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Single Storey Steel Framed Buildings in Fire Boundary Conditions

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FOREWORD

The publication replaces the earlier *Fire and steel construction: The behaviour of steel portal frames in boundary conditions,* which was first published by SCI in 1991. The scope of this publication is now wider and advice is given on additional topics including trusses and lean-to structures.

This publication will assist in the design of single storey industrial buildings which require fire resistance because the building is situated close to the site boundary or to maintain fire compartmentation within the building. It should be of particular interest to Structural Engineers, Architects, Contractors (particularly in design and build) and Building Control Officers.

The authors of this publication are Dr Ian Simms and Mr Gerald Newman of The Steel Construction Institute. Valuable contributions were also made by members of the steering group. The members of the steering group at various times included:

Dr Colin Bailey	BRE
Mr Allan Pottage	Severfield Reeve Structures
Mr John Dowling	Corus
Mr Geoff Harding	DTI
Mr John Brennan	Barrett Steel Buildings
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SUMMARY

Building regulations require that external walls of single storey buildings that are close to the site boundaries should have fire resistance, to at least part of the walls. Any structure that provides support to such walls also has to have fire resistance.

This publication provides, in addition to UK building regulations, design recommendations and guidance for single storey buildings for design in fire situations. Based on earlier research and study, 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 background to the recommendations is given and the mathematical models are explained.

Immeubles à ossature acier d’un seul niveau sous conditions d’incendie

Résumé

Les réglementations relatives aux immeubles stipulant que les murs extérieurs des immeubles à un seul niveau, qui sont situés près de la limite du site de construction doivent posséder une résistance au feu, tout au moins une partie de ces murs. Toute structure qui supporte de tels murs doit également présenter une résistance au feu.

Cette publication fournit, en addition aux règlements UK sur les bâtiments, des recommandations et conseils pour le dimensionnement des immeubles à un seul niveau soumis à un incendie. Basée sur de précédentes études, elle montre que la protection incendie de la structure de la toiture, qui serait coûteuse à mettre en œuvre, n’est pas nécessaire pour autant que les recommandations relatives aux pieds des poteaux soient suivies. Les recommandations de cette publication couvrent les portiques simples et ceux à baies multiples. Les fondements des recommandations sont explicités ainsi que les modèles mathématiques utilisés.

Eingeschossige Stahlbauten bei grenznaher Bebauung

Zusammenfassung

Die Bauvorschriften verlangen, daß Außenwände von eingeschossigen Gebäuden in grenznaher Lage zumindest teilweise eine gewisse Feuerwiderstandsdauer haben sollten. Jede tragende Unterkonstruktion dieser Wände muß ebenfalls eine Feuerwiderstandsdauer haben.

Diese Publikation stellt für eingeschossige Gebäude, zusätzlich zu den Bauvorschriften des Vereinigten Königreichs, Berechnungsempfehlungen und Anleitungen im Brandfall bereit. Auf der Grundlage früherer Forschungen und Studien wird deutlich, daß ein teuerer Brandschutz für die Dachkonstruktion nicht nötig ist, vorausgesetzt, daß Empfehlungen für die Stützenfußausbildung befolgt werden. Die Ratschläge und Empfehlungen beziehen sich auf ein- und mehrschiffige Rahmen, Pultdach-Rahmen, Satteldach-Rahmen und Rahmen mit Fachwerkbindern. Der Hintergrund der Empfehlungen wird dargestellt und die mathematischen Modelle werden erklärt.

Edificios aporticados de una planta con condiciones de contorno de incendio

Resumen

La normativa de construcción de edificios exige que los muros exteriores de edificios de una planta que estén cerca de los contornos del emplazamiento sean resistentes al fuego, por lo menos en parte de aquellos. Cualquier estructura que soporte tales muros debe igualmente ser resistente al fuego.

Esta explicación suministra, complementariamente a la normativa edilicia en U. K., recomendaciones y guía de proyecto para edificios aporticados de una planta en condiciones de incendio.

Basándose en investigaciones y estudios previos demuestra que la costosa protección frente al fuego de la estructura cubierta no es necesaria siempre que se sigan las recomendaciones de proyecto de los pilares. Los consejos se refieren a estructuras de uno o varios vanos, pórticos a dos aguas, estructuras con cerchas de cubierta, etc. Los fundamentos de las especificaciones y de los modelos matemáticos utilizados son explicados cuidadosamente.

Edifici intelaiati in acciaio monocampata in condizioni di limite di incendio

Sommario

Le raccomandazioni per gli edifici richiedono che le pareti esterne degli edifici monopiano vicini ai confini del sito dovrebbero avere, almeno da una parte, adeguata resistenza al fuoco, come pure ogni struttura di sostegno a tali pareti.

Questa pubblicazione fornisce, in aggiunta al regolamento del Regno Unito per gli edifici, criteri ed indicazioni progettuali per edifici monopiano in condizioni di incendio. Sulla base di studi e ricerche iniziali, viene dimostrato che la protezione al fuoco della copertura, che potrebbe essere estremamente costosa, non è necessaria quando vengano rispettati i criteri progettuali per le basi delle colonne. Le informazioni e le raccomandazioni sono relative sia a edifici sia monocampata sia multicampata, a portali con copertura monofalda, a due falde o con copertura reticolare.

Nella pubblicazione viene descritto l'approccio teorico alla base delle raccomandazioni e sono anche contenute spiegazioni sui modelli matematici impiegati.

Brandcellsvillkor för envåningsbyggnader med stålram

Sammanfattning

Byggnadsbestämmelser kräver att ytterväggar som gränsar till brandceller i envåningsbyggnader ska ha brandskydd, åtminstone på delar av väggarna. Likaså måste samtliga konstruktioner som bär upp sådana väggar förses med brandskydd.

Denna publikation kompletterar brittiska byggnadsbestämmelser med rekommendationer och vägledning för utformning av envåningsbyggnader ur brandhänseende. Med hjälp av tidigare forskning och studier visas att det dyrbara brandskyddet på takstrukturen inte är nödvändigt, under förutsättning att man följer rekommendationerna för pelarutformning. Förutom rekommendationer och råd ger publikationen även bakgrunden till rekommendationerna och förklarar de matematiska modellerna.

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 amount of wall requiring fire resistance reduces as the distance from the site boundary increases. In common parlance, if any part of an external wall requires fire resistance, the building is said to have a fire *boundary condition*.

Any structural members supporting such an external wall should also have fire resistance; the supporting members can sometimes include the roof structure. Fire resistance is frequently achieved by applying fire protection to the structural members but fire protection adds to the cost of the building and fire protecting the roof structure is particularly costly.

The principal aim of this publication is to present recommendations that allow the roof structure to be left unprotected whilst meeting requirement of providing fire resistance to external walls. The recommendations are generally applicable throughout UK, although the separate regulations and other documents for England and Wales, for Scotland and for 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.

1.1 Building Regulations

1.1.1 England and Wales

In England and Wales, the *Buildings Regulations*[1] contain simple functional requirements. These requirements use words such as *reasonable* and *adequate* but impose no specific limits, for example, in terms of periods of fire resistance. Periods of fire resistance and other quantified requirements are given in *Approved Document B*[2]. 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.

The functional requirements of the 1991 (and later) Building Regulations replaced earlier prescriptive regulations in which the tests of reasonableness and adequateness had no part. Earlier regulations required particular structural elements to have fire resistance and there was little scope for interpretation.

1.1.2 Scotland

In Scotland, requirements are set out in the *Building Standards (Scotland) Regulations*[3]. These replaced earlier prescriptive Building Standards. A set of *Technical Standards*[4] 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.

1.1.3 Northern Ireland

The Building Regulations (Northern Ireland)[5] express their requirements in terms of performance rather than prescribed methods and standards. *Technical Booklet E*[6] 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.

1.2 Boundary conditions and protected areas

The external walls of a building will require fire resistance will depending on the proximity of a building to a site boundary. Radiation is the principal mechanism of fire spread and the objective of the regulations is to limit the amount of radiation reaching the adjacent buildings by using the external walls as a barrier. This is achieved by limiting the amount of any external wall that does not have fire resistance (unprotected area). The remaining area, (protected area) must have fire resistance in terms of stability, integrity and insulation.

As the distance from the site boundary increases, the area of the external walls that requires fire resistance decreases. To allow the wall to be totally unprotected (i.e., to have 100% of its area unprotected), the building will have to be situated at least 15 m from the boundary (or up to at least 25 m in some cases, depending on usage and size).

For England and Wales, Approved Document B provides simple tabular guidance on small buildings that are close to boundaries. In most cases, it will be necessary to refer to more comprehensive guidance published by BRE[7]. The simple method for small buildings (height not more than 10 m) is reproduced in Appendix C.

The requirements for boundary distances and unprotected areas in the Scottish Technical Standards are very similar to those of England and Wales. The calculation of boundary distance and acceptable unprotected areas is based on the methods given in BRE Report 187[7]. The *Technical Standards* contain a simple method for buildings where the wall facing the boundary is less than 9 m high and less than 24 m long.

Northern Ireland adopts the same approach as England and Wales.

1.3 The boundary condition problem

If the building is constructed close to the site boundary, the external wall of the building will require a degree of fire resistance. It is generally accepted that structural members that support these walls will also require fire resistance to ensure that the walls remain stable for a reasonable period during a fire

The requirement for external walls to have fire resistance has resulted in portal framed buildings being treated as a special case. The argument put forward by regulatory authorities was that the columns and rafters of a portal frame are designed as a single continuous element and as such, the whole element requires the same level of fire resistance. Obviously, providing fire resistance to rafters as well as columns represents a significant increase in cost and therefore alternative solutions were investigated.

Following a study[8] of portal frames in actual fires it was concluded that the most viable alternative to fire protecting rafters is to ensure that, even if the roof collapsed, the stability of the external walls would be maintained. Guidance on how this can be achieved was initially published in 1979 by CONSTRADO[9]. The guide was later republished by SCI, as a second edition, as *The behaviour of steel portal frames in boundary conditions* (P087)[10]. That publication has been referenced in Approved Document B for a number of years and is also mentioned in the *Scottish Technical Standards* and in Northern Ireland in *Technical Booklet E*, as an accepted alternative to fire protection of the whole of a portal frame in boundary conditions.

Designing columns and foundations on boundary walls to resist the forces and moments, as recommended in P087[10], is generally a more economic solution than fire protecting the portal rafter. The engineering solution to the boundary condition problem is considered to meet the “reasonableness” test of the Building Regulations.

However, with the approach now adopted by the regulatory authorities of publishing non-mandatory guidance, it is possible for local authorities to question the “reasonableness” of solutions presented for other types of single storey buildings, such as those utilising trusses. In the past, while the effect that the collapse of a portal rafter would have on the stability of a boundary wall needed to be considered, the mechanism of collapse of other forms of roof construction did not need to be considered. This is because under the old regulations, only elements of structure could require fire resistance and roof members were not considered ‘elements of structure’. This approach is now open to question under the “test of reasonableness” and regulatory authorities may ask designers to consider the consequences of roof collapse in their designs.

1.4 Sprinklers

In buildings fitted with a sprinkler system meeting the requirements of BS 5306-2[11], for the relevant occupancy rating together with the additional requirements for life safety, the risk of fire spreading to adjacent buildings is reduced. The regulatory authorities recognise this reduction in different ways.

1.4.1 Use of sprinklers in England and Wales

The reduced risk of fire spread is recognised by Approved Document B. The boundary distance for a building with sprinklers may be half that required for a building without sprinklers, or alternatively the unprotected area in the boundary wall can be doubled.

More significantly, it is stated in Approved Document B that if the building is fitted with a sprinkler system, the recommendations of P087 to design of the foundation to resist the overturning moment from the collapse of the roof need not be followed.

It reasonable to assume that the same relaxation should apply to the recommendations of this publication.

1.4.2 Use of sprinklers in Scotland

In Scotland, relaxations on boundary conditions are permitted if the building is fitted with a sprinkler system meeting the requirements of BS 5306-2. However, the requirements are more complex than for England and Wales and readers are advised to consult the *Technical Standards*[4].

The *Technical Standard* does not state explicitly that fitting sprinklers removes the need to design the column foundations to resist overturning. Although, the acceptance of P087 in the *Technical Standard* indicates that this is considered to be a reasonable approach, it is up to local authorities to grant relaxations to the regulations on an individual basis.

1.4.3 Use of sprinklers in Northern Ireland

Technical Booklet E recommends that in buildings fitted with a sprinkler system meeting the requirements of BS 5306-2, the boundary distance may be half the distance required for a building without sprinklers or the unprotected area in the boundary wall can be doubled.

Unlike in England and Wales, no explicate statement is made as to the need to design the foundations to resist overturning moments when the building is fitted with a sprinkler system.

1.5 Scope of this publication

The principal aim of this publication is to present a design procedure for portal frames and for frames constructed using trusses that will not require fire protection of the roof structure, for buildings in boundary conditions.

Methods presented, demonstrate the calculation of foundation overturning moments, should the unprotected roof collapse in fire. Guidance is also given on aspects of the maintenance of compartmentation and detailing.

Background information on the behaviour of various types of single storey buildings in fire that has led to the recommendations, is given. Two design examples are also presented.

1.6 Layout of the publication

General recommendations are given in Section 2, based on studies of the behaviour of portal frames in fire and the development of collapse models. The core recommendations remain largely unchanged from those set out in earlier Constrado and SCI publications, apart from some minor modifications in keeping with current building regulations. Section 3 contains Guidance Notes which provide advice to designers on the appropriate application of the general recommendations and how to treat some special cases such as gable frames and compartment walls.

Sections 5 to 9 summarise the background to the recommendations and guidance. Sections 5, explains the basis for reducing the loading that needs to be considered in fire. Sections 6 and 7 describe the behaviour of portal frames with symmetrically pitched rafters and the behaviour of trusses and lattice rafters respectively. Sections 8 and 9 illustrate the requirements of two basic frame types presented using a series of design scenarios. The scenarios relate to possible configurations of boundary conditions and compartmentation.

The mathematical models for calculating overturning moments are given in Appendices A and B.

2 GENERAL RECOMMENDATIONS

This Section presents recommendations which, if followed, should allow main roof members of single storey buildings, in a variety of structural types, to remain unprotected. Readers are recommended to follow the step-by-step procedure, as this will ensure that important parameters are not overlooked.

These recommendations will ensure the stability of the boundary wall without needing to fire protect the roof structure. This is achieved by designing the column foundations to resist the overturning moment that will occur when the roof collapses. The recommendations cover the calculation of the overturning moments, the design of the base to resist this moment, the longitudinal stability of the building and the fire protection requirements for the columns.

2.1 Design procedure

The following basic steps should be followed when designing a single storey building for fire.

Step	Action	Section
1	Determine whether a boundary condition exists.	2.2
2	Determine whether sprinklers will be installed. (Special precautions may not be necessary if sprinklers are present.)	2.3
3	Determine the load on the rafter at the time of the fire.	2.4
4	Calculate the overturning moment due to the collapse of the roof.	2.5
5	Check the resistance of the columns and bases	2.6
6	Design the foundation to resist the overturning moment.	2.7
7	Check the longitudinal stability of the building.	2.8
8	Check the fire protection requirements of columns.	2.9

2.2 Boundary conditions

These recommendations apply to columns supporting protected areas of external walls. Columns supporting protected areas will require fire protection up to eaves level, regardless of the extent of the protected area. Therefore, when less than 100% of the wall is required to have fire resistance it is preferable to arrange the protected area so that it covers as few columns as possible thereby minimising the requirements for fire protection and moment resisting bases.

2.3 Sprinklers

In the event of fire, the effective operation of a sprinkler system would control the size of the fire and may even extinguish the fire. The resulting risk of fire spreading beyond the building of origin is greatly reduced. It follows that in such situations, rafter collapse would be unlikely and the stability of boundary walls would not depend on base fixity.

In England and Wales, if a sprinkler system is fitted, it is recognised by Approved Document B[2] that there is no need to design for the collapse of a roof in fire. In Scotland and Northern Ireland, acceptance of P087[10] as a suitable method of designing portal frames indicates that this is considered to be a reasonable approach (See Section1.4).

2.4 Loading

It is recommended that the overturning moment resulting from the collapse of the roof structure in fire be based on a reduced design load for the fire limit state. The design load should be calculated using the load factors given in BS 5950-8[12].

2.4.1 Dead load

The dead load will be due to the self-weight of the structural members and the cladding. In a fire of sufficient intensity to cause collapse of the roof structure, it may be assumed that some of the cladding will be destroyed by the fire. The extent of this damage depends on the type of cladding. Table 2.1 gives a guide to the typical reduction in cladding self-weight due to fire, depending on the type of cladding system used.

2.4.2 Wind load

The duration of a fire is usually measured in hours. The likelihood of a fire occurring simultaneously with high winds is therefore unlikely and it is recommended that the effect of wind loading be ignored when considering the stability of boundary walls.

Note that an amendment to BS 5950-8, due to be published at the end of 2002, is expected to state that, for the purposes of assessing boundary wall stability, wind loading may be ignored.

2.4.3 Service load

In fire, the load due to services should be taken as one third of the load used for normal limit state design. This makes allowance both for the assumed services not being present in the first instance and for some services becoming detached during the fire.

2.4.4 Imposed load

Snow forms the principal imposed roof load for single storey buildings. The combination of snow and fire are remote. BS 5950-8 states that snow loading may be ignored. (See Section 4.2).

Table 2.1 *Percentage dead weight of roof cladding systems remaining at the time of the rafter collapse*

Inner Lining		Insulation		Outer Covering	
Mineral insulation board	100%	Glass or mineral fibre	100%	Steel	100%
		Thermoplastic foams	0%	Aluminium	100%
		Thermosetting foams	70%	Fibre cement	100%
Plaster board	0%	Bonded thermoplastic foams	0%	Steel	100%
		Glass or mineral fibre	0%	Aluminium	10%
		Unbonded foams	0%	Fibre cement	10%
Plaster board	50%	Bonded thermosetting foams	50%	Steel	100%
				Aluminium	50%
				Fibre cement	50%
Steel	100%	Glass or mineral fibre	100%	Steel	100%
		Thermoplastic foams	0%	Aluminium	100%
		Thermosetting foams	70%	Fibre cement	70%
Steel	100%	Fibre insulating board	70%	Steel	100%
				Aluminium	100%
				Fibre cement	100%
Thin linings e.g. foil, embossed papers and plastics	0%	Mineral or glass fibre	0%	Steel	100%
		Most foamed plastics	0%	Aluminium	0%
				Fibre cement	10%
Aluminium	0%	Phenolic foams	50%	Steel	100%
				Aluminium	50%
				Fibre cement	50%
Fibre cement sheets	10%	Unbonded glass or mineral fibre	10%	Fibre cement	10%
		Thermosetting foams	10%		
		Thermoplastic foams	0%		
Factory assembled bonded systems					
Foil, paper or plastic facings	0%	Urethane or isocyanurate foams	70%	Steel	100%
		Phenolics	80%	Aluminium	100%
Steel	100%	Thermosetting foams	80%	Any	100%
Aluminium	0%	Thermosetting foams	80%	Any	100%
Roof lights					
Single skin plastic	0%				
Double skin plastic	50%				

Note: Asbestos cement sheets are assumed to behave in a similar manner to fibre cement sheets.

2.5 Overturning moments

2.5.1 Symmetrical pitched portal frames

A full mathematical model of the collapse mechanism for symmetrical pitched portal frames has been developed and is presented in Appendix A. The use of the full model is rather tedious for calculations by hand. Consequently, a simplified model has been developed. The model forms the basis of the recommendations given below. These recommendations apply to symmetrical pitched rafters of single and multi-bay frames. The rafters may or may not have haunches. The columns in the boundary wall must be adequately restrained in the longitudinal direction.

The requirements for fire protection of columns and the requirement to resist the calculated forces and moments on the column bases are applicable only to columns that support a protected area of boundary wall. Columns which do not support protected areas will not require fire protection or moment resisting column bases. Thus, when only one side of the building has a boundary condition, the columns on the non-boundary side do not require protection or resistance to the forces calculated according to these expressions. The expressions are derived assuming symmetrical conditions (including fire protection to the columns) and are conservative when applied to a frame where the non-boundary column is unprotected and/or not designed to resist the overturning moment.

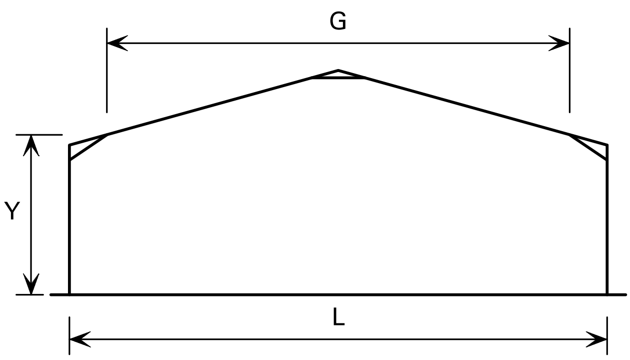


Figure 2.1 *Frame dimensions used in calculation of overturning moment.*

Column and column base

The foundation, column and column base should be designed to resist the following moment and reactions.

Vertical Reaction:

$$V_R = \tfrac{1}{2} w_f SL + W_D$$

Horizontal Reaction:

$$H_R = K\left(w_f SGA - \frac{CM_P}{G} \right) \text{but not less than } \frac{M_C}{10Y}$$

Overturning moment:

$$OTM = K\left[w_f SGY\left(A + \frac{B}{Y} \right) - M_P\left(\frac{CY}{G} - 0.065 \right) \right] \text{but not less than } \frac{M_C}{10}$$

in which $B = \dfrac{L^2 - G^2}{8G}$

where:

w_f is the load at the time of collapse (kN/m^2)

W_D is the dead load of the wall cladding (kN)

S is the distance between frame centres (m)

G is the distance between ends of haunches (m)

Y is the vertical height of end of haunch (m)

M_P is the plastic moment of resistance of the rafter (kNm)

M_C is the plastic moment of resistance of the column (kNm)

K = 1 for single bay frames or is taken from Table 2.3 for multi-bay frames

L is the span (m)

A and C are frame geometry parameters, given in Table 2.2

Table 2.2 *Frame geometry parameters A and C*

a) Span/height ratio greater than 2:1

	Rafter pitch										
	0°	**3°**	**6°**	**9°**	**12°**	**15°**	**18°**	**21°**	**24°**	**27°**	**30°**
A	1.01	0.99	0.93	0.85	0.76	0.68	0.61	0.54	0.49	0.44	0.40
C	1.05	1.02	0.96	0.88	0.79	0.70	0.62	0.56	0.50	0.45	0.41

b) Span/height ratio between 1:1 and 2:1

	Rafter pitch										
	0°	**3°**	**6°**	**9°**	**12°**	**15°**	**18°**	**21°**	**24°**	**27°**	**30°**
A	0.87	0.85	0.81	0.76	0.69	0.63	0.57	0.52	0.47	0.42	0.47
C	0.91	0.90	0.85	0.79	0.73	0.66	0.60	0.54	0.49	0.44	0.49

2.5.2 Multi-bay portal frames

The behaviour of multi-bay frames is very similar to that of single-bay frames. Therefore, the overturning moment on the columns supporting the boundary wall can be calculated using the calculation method given in Section 2.5.1 and using the appropriate multiplication factor, K, from Table 2.3.

The use of this multiplication factor takes account of the additional moment that will occur on the boundary columns of a tall multi-bay frame due to the collapse of unprotected internal columns adjacent to the boundary.

The multiplication factor is only applied if an unprotected column supports a rafter that spans to an affected edge column. It is not applied when considering protected columns or columns more than one bay from a boundary wall. If the span:height ratio of the frame is less than the lower limit given in Table 2.3 the internal column adjacent to the boundary must be fire protected.

Table 2.3 *Multiplication factor for multi-bay frames*

Pitch (not greater than)	Range of span/height ratio	Multiplication factor K
3°	2.5 and over	1.0
	1.7 to 2.5	1.3
6°	2.3 and over	1.0
	1.6 to 2.3	1.3
9°	2.1 and over	1.0
	1.6 to 2.1	1.3
12°	1.8 and over	1.0
	1.6 to 1.8	1.3
Over 12°	1.6 and over	1.0

2.5.3 Tapered rafters

These recommendations apply to steel portal frames with tapered rafters where:

- The frame is constructed from welded plates and both the column and rafter may be tapered.
- The frame may or may not have haunches.
- The frame may be single or multi-bay.
- The rafter adjacent to the boundary must be symmetrical about the centre of its span.

Column and column base

The foundations, column and column base should be designed to resist the overturning moment and reactions calculated using the full mathematical model described in Appendix A, with the following minor modifications.

The fire hinge near the eaves is assumed to occur:

> Midway between the first and second stays restraining the lower flange
>
> or
>
> at a distance, along the lower rafter flange, measured from the intersection of the inner column flange and the lower rafter flange, equal to 2.5 times the rafter depth at the intersection point.

The hinge location is taken at whichever point is nearest to the eaves.

As an alternative to the above, for ease of computation, the hinge may be considered to be at the intersection of the inner column flange and the lower rafter flange. The calculation is then conservative. (Dimensions X_1, X_2, R_1 and R_2 in the calculation are modified accordingly.)

M_{P1} should be based on 85% of the normal plastic moment of resistance of the rafter at the hinge point, multiplied by 0.065.

M_{P2} should be based on the full plastic moment of resistance of the rafter at the apex, multiplied by 0.065.

For multi-bay frames, the multiplication factor, K, taken from Table 2.3, should be applied to the moment and horizontal reactions calculated using this method.

The minimum values for H_R and OTM should be taken as 10% M_C.

2.5.4 Mono pitched frames

Mathematical model of rafter collapse

It is not possible to devise a simplified model for mono pitch portal frames, in the way that it was for symmetrical pitched portal frames. Consequently, a full model similar to that given in Appendix A (for duo pitched frames) has to be used. The model is based on the frame layout shown in Figure 2.2.

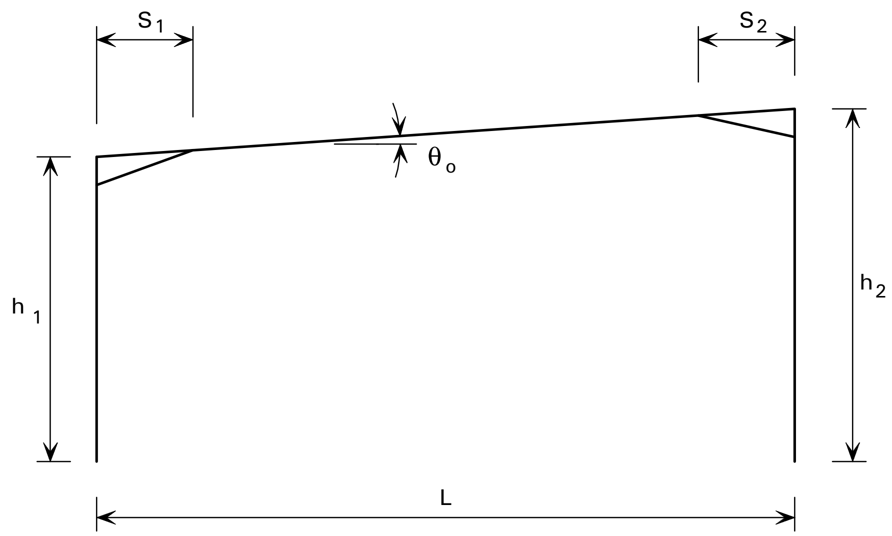


Figure 2.2 *Basic monopitch frame*

The overturning moment is calculated for a frame with an inverted rafter, as shown in Figure 2.3. The rafter is assumed to collapse with three fire hinges located at the end of each haunch and at the mid span of the rafter.

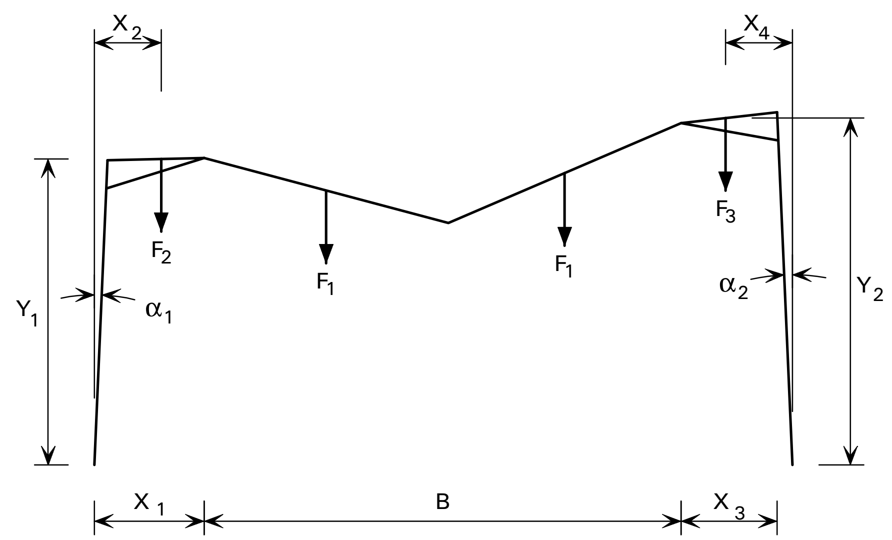


Figure 2.3 *Collapsed monopitch frame*

Overturning moment

The moments on the bases of the columns are given by:

$$OTM_1 = HY_1 + M + F_2 X_2 + (F_1 - v) X_1$$

$$OTM_2 = HY_2 + M + F_3 X_4 + (F_1 + v) X_3$$

where:

H is the horizontal reaction acting at the end of the haunch

Y_1, Y_2 are the heights to the end of the haunch

X_1, X_3 are the horizontal dimensions between the base of the column and the end of the haunch

X_2, X_4 are the horizontal dimensions between the base of the column and the centre of the haunch

M is the moment resistance of the rafter in fire

F_1, F_2, F_3 are the force acting on the rafter and haunches due to gravity loads

v is the vertical shear force at mid-span (see Figure 2.4)

The calculation of these forces and dimensions are described as follows.

Horizontal reaction

Considering the moments and forces shown in Figure 2.4, for vertical equilibrium:

$$Q + P = 2F_1$$

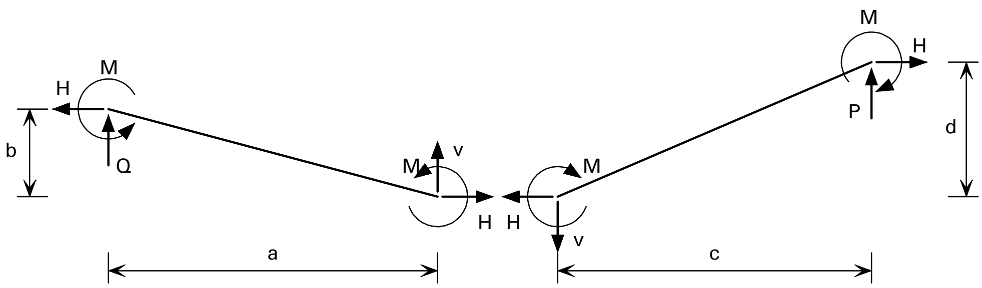


Figure 2.4 *Forces and moments on rafter*

Considering the equilibrium of the rafter to the left of the central hinge and taking moments about the central hinges gives:

$$Hb + \frac{F_1 a}{2} + 2M = Qa$$

Repeating this process for the rafter to the right gives:

$$Hd + \frac{F_1 c}{2} + 2M = Pc$$

As the two halves of the rafter are in equilibrium, the H on both sides must be the same and the previous two equations can be rearranged to give.

$$H = \frac{F_1\, c - 2M\left(1 + \frac{c}{a}\right)}{d + \frac{bc}{a}}$$

The dimensions a, b, c, d are the vertical and horizontal dimensions describing the location of the rafter fire hinges. These dimensions are calculated as described below.

Vertical shear on rafter at central hinge position

Referring to Figure 2.4 and taking moments about the left hand end of the rafter, the vertical shear on the rafter at the central hinge is given as follows.

$$v = \frac{F_1}{2} + \frac{H\,b}{a} - \frac{2M}{a}$$

Forces on rafters

The force, F_1, in the rafter between fire hinges is given as follows.

$$F_1 = \tfrac{1}{2} wz\left[L - \cos q_o\,(S_1 + S_2)\right]$$

The forces, F_2 and F_3, acting at the centre of each haunch are given as follows.

$$F_2 = wz\,S_1 \cos q_o$$

$$F_3 = wz\,S_2 \cos q_o$$

Where z is the frame spacing.

Frame dimensions

Considering the frame geometry shown in Figure 2.2 and Figure 2.3, the following dimensions can be derived.

Height of the end of the haunches Y_1 and Y_2

$$Y_1 = h_1 \cos a_1 + S_1 \sin(q_o - a_1)$$

$$Y_2 = h_2 \cos a_2 - S_2 \sin(q_o + a_2)$$

Horizontal dimensions to the ends of the haunches

Horizontal lengths from column base to end of haunch X_1 and X_3

$$X_1 = h_1 \sin a_1 + S_1 \cos(q_o - a_1)$$

$$X_3 = h_2 \sin a_2 + S_2 \cos(q_o + a_2)$$

Horizontal distance from column bases to centres of haunches

$$X_2 = h_1 \sin a_1 + \frac{S_1 \cos(q_o - a_1)}{2}$$

$$X_4 = h_2 \sin a_2 + \frac{S_2 \cos(q_o + a_2)}{2}$$

Rafter fire hinge positions

Rafter length between ends of haunch, B, including allowance for elongation.

$$B = L - X_1 - X_3$$

Figure 2.5 shows the geometry of the rafter between the ends of the haunches in more detail. The following geometric parameters can be determined.

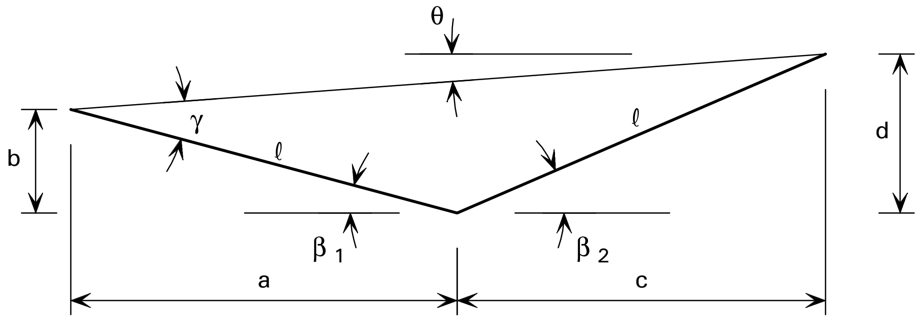


Figure 2.5 *Rafter geometry*

The slope of line joining the ends of the haunches, q is given by:

$$q = \tan^{-1}\left(\frac{Y_2 - Y_1}{B}\right)$$

The length of the rafter between the end of the haunch and the central fire hinge position including allowances for elongation, ℓ is given by:

$$\ell = \frac{1}{2} \times \frac{1.02\left(L - \cos q_o \left(S_1 + S_2\right)\right)}{\cos q_o}$$

The angle, g is given by:

$$g = \cos^{-1}\left(\frac{B}{2\ell \cos q}\right)$$

The rafter sag angles b_1 and b_2 are given by:

$$b_1 = q - g \qquad b_2 = q + g$$

The horizontal and vertical dimensions between the fire hinge positions are given by:

$$a = \ell \cos b_1 \qquad c = \ell \cos b_2$$

$$b = \ell \sin b_1 \qquad d = \ell \sin b_2$$

Assumptions made when using the model

In order to solve the above equations the same assumptions are made as for symmetrical pitched frames.

Rafter length

When loaded at high temperatures, the rafter would strain appreciably. It is reasonable to assume a value of 2% for this strain.

Fire hinge moments

The moment capacity, M, at the fire hinges will be significantly less than the moment capacity at normal temperature. For frames utilising a hot rolled universal sections as rafters, it is assumed that at the time of collapse M is equal to 6.5% of the normal plastic moment of resistance of the rafter. This value represents the residual strength of steel at 890°C.

Column angle

It is assumed that the column deflection angle is one degree. A larger rotation could be assumed, provided that it can be demonstrated that the base can sustain that amount of rotation. It is reasonable to assume that a rotation of one degree can be achieved by elongation of the holding down bolts and some deformation of the base plate. If a larger rotation is assumed, a slightly reduced overturning moment will be obtained.

2.5.5 Trusses and lattice rafters

The overturning moment for column bases in buildings with roof trusses can also be calculated using the simplified method given in Section 2.5.1. However, because of the mode of failure of trusses, the residual moments in the rafter are assumed to be zero and the haunch length is also assumed to be zero.

2.6 Resistance of columns and bases

The tensile capacity of the holding down bolts should be checked under the action of the overturning moment. As the bolts will be located below ground level, the check can be carried out for normal temperatures by using the reduced partial safety factor given below. Similarly, the bending capacity of the base plate should be checked as for normal design but with a reduced safety factor.

Safety factors

For holding down bolts, a partial safety of 1.0 should be used.

For both the column and base plate, a partial safety factor of 1.2 against formation of a plastic hinge should be used.

2.7 Foundations

Bases with a large moment of resistance will be capable of supporting the column in a reasonably upright position but nominally pinned bases are likely to prove inadequate, thus allowing the collapsing rafter to pull the columns over.

If the base has sufficient strength and ductility, it will allow the column to lean inwards to an equilibrium position at which the overturning moment is equal to the moment of resistance of the base. The column will then remain static while the rafter continues to collapse as if the base were fixed.

A fixed-based portal frame will usually have adequate base fixity to resist rafter collapse. The column foundations will not need to be checked if the span/height ratio is greater than 2.0.

It is not intended to provide detailed advice on foundation design. The solution chosen for in particular situation will depend on a variety of factors. However, the following guidance might be helpful to designers.

- Problems arise because large overturning moments are associated with small vertical loads. Therefore, do not underestimate the contribution to vertical load from cladding or dwarf walls.
- In some cases, it is possible to assume that as the column leans inwards and bears against the floor slab, the bottom of the foundation tends to kick outwards. If ground conditions allow it, and some horizontal resistance can be relied upon from the soil, then a proportion of the moment can be resisted in this way, as shown in Figure 2.6. The moment, M, applied to the column is resisted by the moment, M_R, due to bearing under the foundation and the moment generated by the horizontal reactions, V_H.
- The design is based on ultimate conditions. It is very unlikely that they will ever exist. It is therefore reasonable to use every available means to resist overturning. Re-use of the foundations should not be a design consideration.

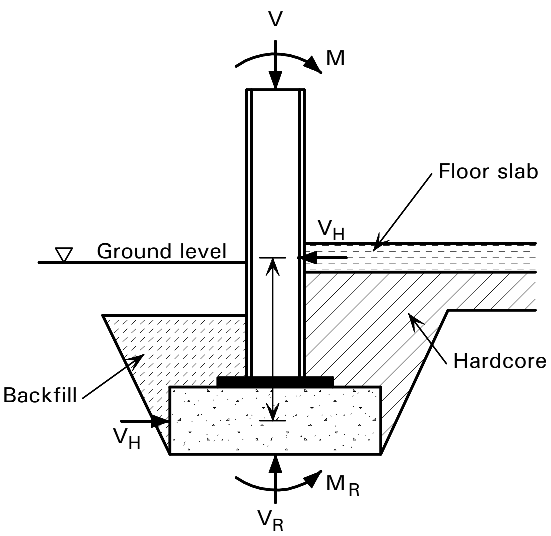


Figure 2.6 *Horizontal resistance of foundation*

2.8 Longitudinal stability

It is important that sufficient longitudinal stability exists in order to ensure that the integrity of the boundary walls is not jeopardised by out of plane deformation of the structural frames. Evidence suggests that conventionally designed portal frames have adequate restraints to resist longitudinal deflections that could lead to premature collapse in fire.

The requirement for adequate longitudinal stability would be met by providing adequate base fixity and adequate restraint to the column.

2.8.1 Base fixity in the longitudinal direction

The base plate of each column should be connected to the foundation by at least four equal diameter holding down bolts. These bolts should be spaced symmetrically about the minor axis of the column at a minimum spacing equal to 70% of the flange width. It may be cost effective to offset the bolts towards the tension side, although this has no effect on the base fixity.

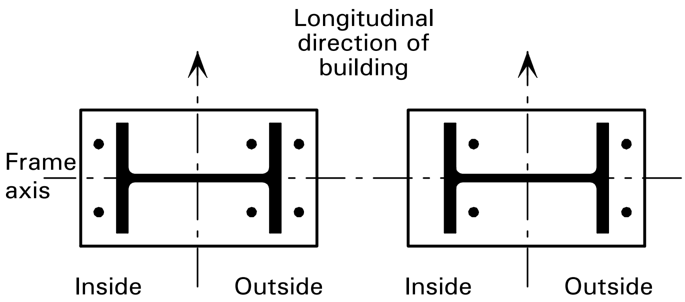


Figure 2.7 *Possible arrangements of offset holding down bolts*

The bolt arrangements in Figure 2.7 are both acceptable.

Alternatively, the columns should be concreted into the base, if the concrete base is capable of resisting an overturning moment in the longitudinal direction equal to that resisted by the holding down bolts in the requirement above.

2.8.2 Longitudinal restraint to column

Where each column is connected to a masonry wall that is assumed to restrain the column in the plane of the wall at normal temperatures, the column can also be assumed to have adequate longitudinal restraint in fire, provided that the height of the masonry wall is not less than 75% of the height to eaves.

If the frame and horizontal members restraining the column are designed to the appropriate part of BS 5950, this will also be adequate for fire. The horizontal members do not require fire protection.

Alternatively, restraint can be provided in the area above any protected area of wall by horizontal steel members having a combined tensile strength of not less than:

$$2.5\% \text{ of } V_R \times \sum \frac{\textit{height of any unprotected area}}{\textit{height of eaves}}$$

in which the summation is over the number of frames.

A protected area of wall would be a masonry wall connected to the column or an insulated clad wall in which the horizontal members are fire-protected to the same standard as the wall.

The design strength of an unprotected member may be assumed to be 0.065 of the design strength for normal temperature design.

2.9 Fire protection

For all types of building, affected boundary columns should be fire-protected up to the underside of the haunch, or to the rafter if no haunch exists. The fire-protected columns should have the same fire resistance as required for the wall. It is not necessary to protect stays restraining the inner flange of a column.

The period of fire resistance required for the wall can be determined from *Approved Document B*[2] .

3 GUIDANCE NOTES

These guidance notes have been developed to provide standardised practical solutions to a number of issues that are frequently raised by designers of single storey buildings.

3.1 Gables

Where a boundary condition exists on the gable wall of a building, the requirement to consider the stability of the wall is the same as for any other elevation.

Figure 3.1 shows a typical arrangement of a gable frame for a portal frame building. The steelwork providing support to the gable wall should be designed to have the same standard of fire resistance as the boundary wall. In most cases, this means that the columns in the gable frame will require fire protection but, in the case of masonry walls, may also mean that the rafter will need protection if the wall is tied to it for stability.

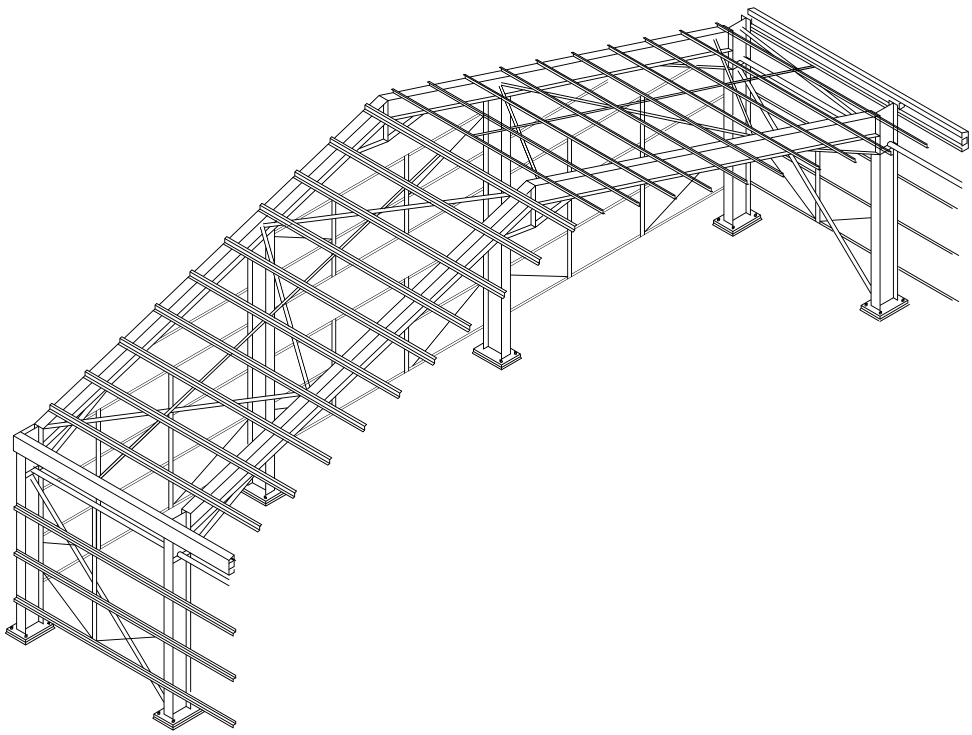


Figure 3.1 *Typical arrangement of a gable frame in a portal framed building.*

However, for single storey buildings, and especially for portal frames, even when a gable frame design includes fire-protected columns, building control authorities often question the reasonableness of the design if it does not consider the stability of the gable columns under the action of a notional horizontal force resulting from the collapse of the roof structure.

Under pre-1991 building regulations, the designer did not have to consider explicitly what might happen in a fire. Therefore, fire protecting the gable columns when the gable wall was close to a site boundary was considered to be sufficient. These columns were classed as elements of structure, but no account had to be taken of the effect of the behaviour of non-structural elements attached to these columns. Therefore, as non-structural elements, the collapse of the roof members did not affect the design of the gable frame.

Now that the requirements of building regulations have been relaxed to permit the acceptance of reasonable designs, not all regulating authorities accept this approach and designers are often asked to consider the lateral stability of the gable columns in boundary conditions. The design scenario being considered is shown in Figure 3.2. As the roof structure collapses, it exerts an out of plane horizontal force on the gable frame at roof level.

As the magnitude of the horizontal force would be expected to be small and is spread over a number of columns it is sufficient in these circumstances to provide a reasonably robust base to gable posts in order to ensure the stability of the boundary wall. This can be achieved by designing the base to resist 10% of the plastic moment of resistance of the post. This is likely to result in the column having a sensibly proportioned four-bolt base plate connection. The moment considered to act on the column is assumed to be caused by a horizontal force pulling inwards at the top of the post.

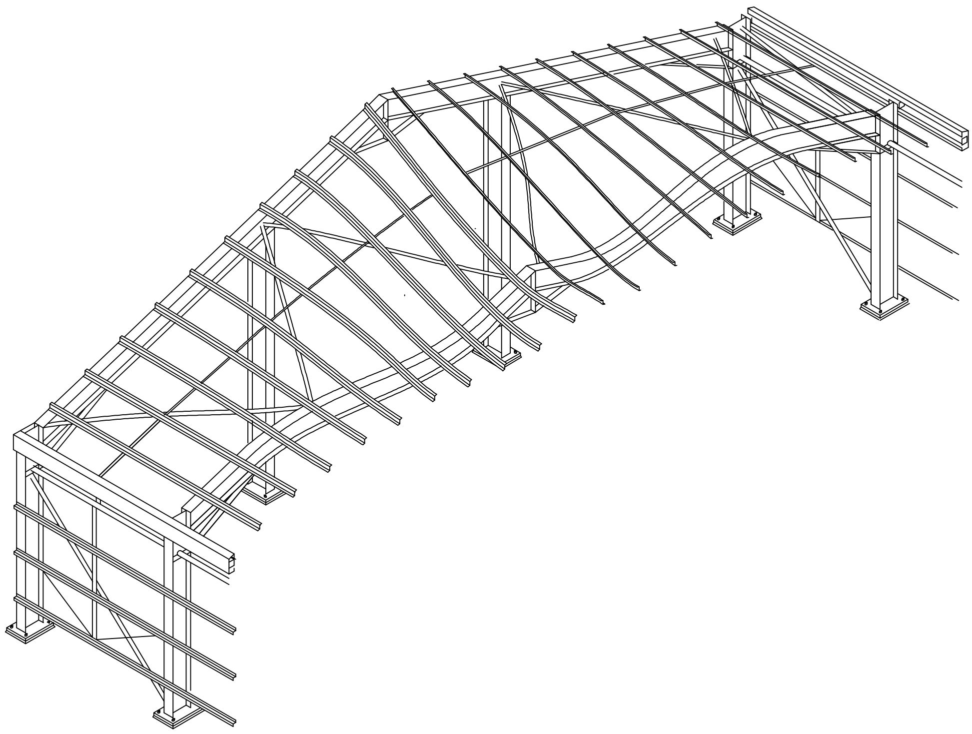


Figure 3.2 *Deformation of the roof structure adjacent to a gable during a fire*

3.2 Cladding

3.2.1 Composite cladding panels

There are a number of insulated cladding systems available on the market. These consist of internal and external metal facings with a core of insulation. The insulation used is typically polyurethane, polyisocyanurate or mineral wool. The performance of the cladding in terms of surface spread of flame, contribution to fire growth and fire resistance needs to be considered when selecting a cladding system. For guidance, see manufacturers' data or for more generic advice consult *The LPC Design Guide*[13]. Product approval to LPS 1181[14] is a good indication of all round fire performance.

In the construction of cold stores, it is common to use composite panels with thermoplastic cores such as expanded polystyrene. In fire, thermoplastics will burn freely, contributing to the fire growth and flame spread. The loss of bond between the facings usually results in structural collapse of the panels. This has recently been covered in Appendix F of Approved Document B (2000 edition).

3.2.2 Eaves beam

Designers are often unclear as to whether the eaves beam requires fire protection. It is common for manufacturers of fire rated cladding systems to advise fire protecting the eaves beam. This is probably a conservative approach as it is likely that the connection between the eaves beam and roof cladding would provide sufficient support to the eaves beam in fire, even if unprotected. Approved Document B does make fire protection of the eaves beam a specific recommendation.

3.2.3 Sheeting Rails

It is common for cladding to be tested for fire resistance on the basis of a 3 m x 3 m specimen. Under these conditions, it is necessary to use slotted holes in the connecting cleat in order to relieve the thermal stress in the sheeting rails. As the fire resistance of the product has been assessed by testing a specimen with slotted holes, it is often the case that the product is then specified with slotted holes for this connection. However, in a real building the degree of restraint is probably less and the ability to deform is greater compared to the situation in a fire test and SCI feel that there is, in practical terms, little to be gained by specifying slotted holes and that ordinary clearance holes are adequate.

3.3 Lean-to Structures

Single storey buildings are often constructed with a portal frame and a lean-to structure that relies in part on the main frame for stability, as shown in Figure 3.3. The structure considered consists of a symmetrical pitched portal frame with a mono-pitched lean-to structure on one side. The mono pitched rafter R2 is connected to the columns C2 and C3 with pinned or moment connections. The building may also be divided into fire compartments using one or more compartment walls.

This guidance note has been prepared following detailed consideration of a number of design scenarios. The design scenarios are described in Section 8 and may help illustrate the advice contained in these guidance notes.

3.3.1 Boundary walls

Where the columns of a portal frame support a boundary wall, they should be fire-protected to the same standard as the wall they support and the column base and foundation should be designed to resist the overturning moment due to the collapse of the roof structure.

The overturning moment should be calculated using the appropriate guidance given in Sections 2.5.1, 2.5.4 or 2.5.5, depending on the type of structure being designed.

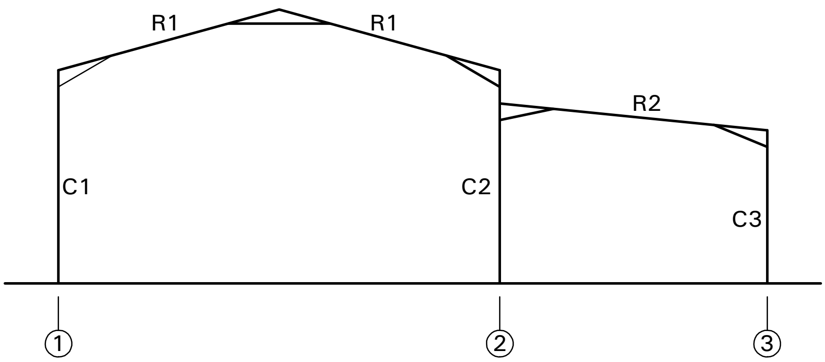


Figure 3.3 *Basic portal frame with lean-to section*

3.3.2 Compartment walls

In buildings where either the main portal rafter or the rafter of the lean-to structure span over a compartment wall, provisions must be made to ensure that the performance of the compartment wall and the stability of the cold structure are not affected by the collapse of the fire affected rafter.

Calculation of horizontal reaction from rafter collapse

If only part of the rafter is heated, because of the presence of the compartment wall, the methods of calculating the horizontal reaction need to be adapted to suit the particular structural arrangement. If the compartment wall crosses a symmetrical pitched rafter, the horizontal reaction caused by the collapse of the portion of rafter on either side of the wall should be calculated using the rules for mono pitched rafters. When the heated section of rafter includes the apex, the horizontal force should be calculated based on a monopitch frame that has the same rafter length as the length of the pitched rafter, between the eaves and the top of the compartment wall, as shown in Figure 3.4.

Design of rafter prop

An effective way of supporting the rafter in the line of the compartment wall is to provide a rafter prop. It is intended that this prop is only effective in the fire condition. For initial sizing purposes, the rafter prop should be designed for a gravity load equal to half the fire design load on the rafter. When calculating the fire design load, the reduced dead load applies only to areas of the roof affected by fire. When checking the stability of a cold frame, the force in the prop may increase depending on whether or not the column bases are designed to be moment resisting for the fire limit state.

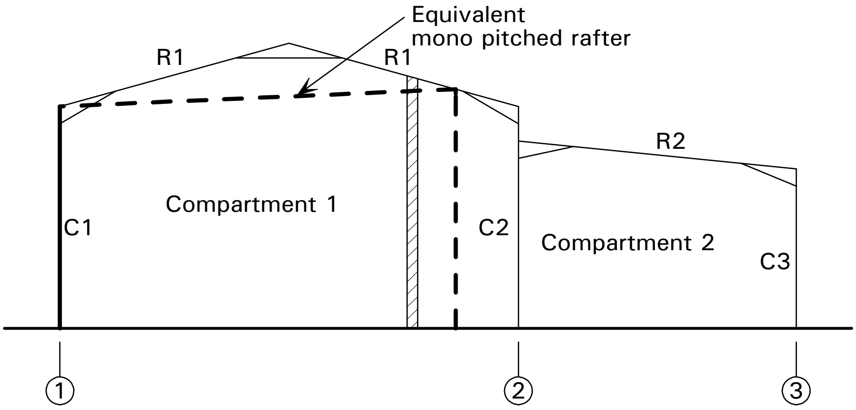


Figure 3.4 *Equivalent mono pitched rafter.*

Stability of the cold structure

The loss of continuity in all or part of a main portal rafter when there is a fire in one compartment will result in an additional horizontal and vertical reaction on the cold part of the structure. The stability of the cold structure under the action of these forces must be considered. The three main concerns are: that any loss of stability will cause disproportionate collapse beyond the fire compartment; that loss of integrity in the compartment wall will allow the fire to spread within the building; and that collapse of the boundary wall will allow fire to spread externally.

3.4 Two storey sections

A common variation in single storey buildings is the inclusion of a two storey section, usually for office accommodation, as shown in Figure 3.5. This two storey section is partitioned from the main single storey part of the building with a fire-separating wall.

This guidance note has been prepared following detailed consideration of a number of design scenarios. The design scenarios are described in Section 9 and may help illustrate the advice contained in these guidance notes.

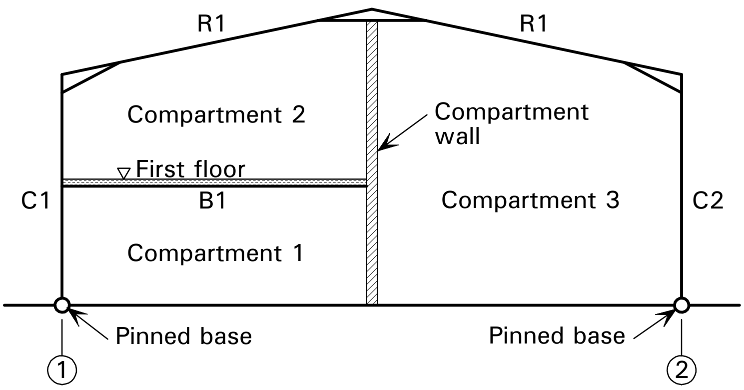


Figure 3.5 *Single storey building with a two storey section*

3.4.1 Boundary walls

Where the columns of a portal frame support a boundary wall they should be fire-protected to the same standard as the wall they support. Also, the column base and foundation should be designed to resist the overturning moment due to the collapse of the roof structure.

The overturning moment should be calculated using the appropriate guidance given in Sections 2.5.1, 2.5.4 or 2.5.5, depending on the type of structure being designed.

3.4.2 Calculation of horizontal reactions from the fire-affected rafters

When single storey buildings contain compartment walls running perpendicular to the span of the frame, a fire will only heat part of the rafter at a time. Therefore, it can be assumed that the horizontal reaction due to collapse of the fire-affected rafter will be lower than if the whole rafter were heated simultaneously.

The calculation method for mono pitched frames should be used to calculate the horizontal reactions for the rafter on either side of the compartment wall. This method will result in lower values of horizontal reaction.

Conservatively, the horizontal reactions may still be determined by assuming that the whole of the rafter is heated simultaneously and employing the calculation method for symmetrical pitched rafters.

3.4.3 Fire protection of portal columns

The extent of the fire protection provided to the portal columns could reasonably be varied, depending on the reasons for providing the protection in the first instance. If the column is protected because it is providing support to an external boundary wall, the fire protection should be provided over the full height of the column.

However, if the protection is provided to the column where it supports the first floor of a two storey section only, it would be reasonable to fire protect the column up to the level of the first floor only.

3.4.4 Stability of cold structure

In cases where only part of a continuous roof structure is affected by fire, the designer must consider the stability of the remaining cold structure. At the ends of the fire-affected rafter, a horizontal and vertical reaction will be transmitted to the cold frame in addition to its normal gravity loading. The frame must be checked to ensure that the loading will not cause of a plastic hinge to form in the rafter or column and that the structure will not be pulled over. The typical loading is shown in Figure 3.6.

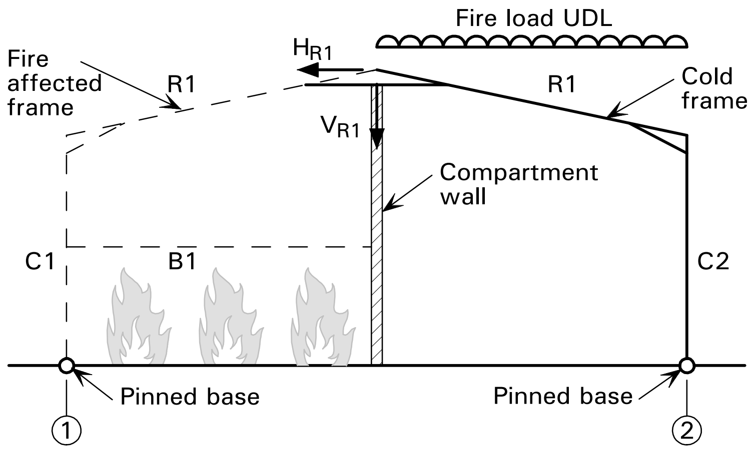


Figure 3.6 *Stability of cold structure*

3.5 Propped portals

Portal frames are occasionally designed with intermediate props to the portal rafter. This enables the designer to reduce the size of the section required for the rafter in normal cold design. In fire situations, these props, which are usually fairly light column sections, are vulnerable as they tend to heat up and lose strength rapidly. When considering the stability of the boundary columns, propped rafters should be treated in one of two ways.

The first alternative is that the prop is left unprotected. In this case, the overturning moment on the boundary columns is calculated based on the full rafter span, assuming that the prop will collapse completely in fire.

The second alternative is to protect the prop so that it remains stable during the fire and continues to support the rafter. The rafter can then be assumed to act as a mono pitched rafter between the external column and the prop. This will significantly reduce the overturning moment on the external column.

A similar approach should be adopted when considering a frame with more than one prop.

3.6 Tied portals

In a tied portal, the tie is used to reduce eaves spread, thus allowing smaller columns and rafters. The tie, for normal loading, will usually be in tension. In fire, the tie is very vulnerable and, as it will normally be a fairly slender section, it will not be able to resist the inward movement of the eaves as the rafter collapses. The application of fire protection to the tie may not appreciably help because the tie is designed to act in tension and will readily buckle under compressive load.

It is therefore suggested that ties are ignored in fire. This often results in large overturning moments and may adversely influence the initial choice of a tied frame.

3.7 Asymmetric frames and curved rafters

Asymmetric frames may have an off-centre ridge or columns of different heights. For such cases, the overturning moment in boundary conditions can be calculated based on an equivalent symmetric frame.

When the columns are of equal height, an equivalent symmetric frame should be constructed with the same total rafter length and frame span but with a central ridge. In order to make this possible, the rise between eaves and apex will not be the same as the original frame. The moment of resistance of the rafter should be taken as the average value for the asymmetric frame. The overturning moment is then calculated using the procedure for a symmetric frame.

If the asymmetric frame has columns of different heights, then two equivalent symmetric frames should be evaluated, one for each column height. If the rafter is also asymmetric, symmetric rafters of equal length should be used, as before.

Frames with curved rafters should also be checked using an equivalent symmetric duo pitched rafter. The equivalent frame should have the same horizontal span and the same overall rafter length as the original frame. To make this possible, the rise between eaves and apex will not be the same as the original frame.

3.8 Valley beams

It is common in portal frame construction to omit alternate valley columns and support the rafters on a valley beam. In fire, this beam is very lightly loaded and has been observed to perform well. It is not generally considered necessary to fire protect valley beams.

3.9 Ridge beams

In propped portals where the ridge is supported, alternate props are sometimes omitted. The intermediate rafters are supported by a ridge beam which in fire will normally be lightly loaded. Although little actual evidence is available of performance in fires, it is thought that they will behave similarly to valley beams and fire protection is not generally considered necessary.

3.10 Dwarf walls

Many industrial buildings have a dwarf wall of masonry construction about 1 m in height, with cladding above the wall. If the building is situated far enough away from the boundary, the dwarf wall may provide a sufficient amount of protected area; then it may be possible to design the steel portal frame without any fire protection or resistance to overturning moments. The masonry dwarf wall should be designed to be free standing and the masonry should not be built into the web of the column. If the roof structure collapses in a fire, pulling over the columns, the wall will not be disrupted and will continue to perform its function as a fire resisting external wall. In this case, it is reasonable not to provide the column with a ‘fire base’ and not to fire protect the column.

3.11 Internal compartment walls

The stability of internal compartment walls must be maintained during a fire. Observations from actual fires noted in the original study[8] were that where internal walls were parallel to the main structural frames, they preformed well in a fire. It was concluded that the cold structure on one side of the wall could be relied on to support the wall, even though the fire-affected steel on the other side was in a state of collapse.

Therefore, where the plane of a compartment wall is parallel to the plane of the portal frames, the bases of columns supporting the wall will not require any specially fixity but the columns should be properly fire-protected.

Where the internal walls are perpendicular to the span of the main structural frames, collapse of the roof structure can affect the stability of the compartment wall. If the rafter/truss is affected by fire on one side, the resulting loss of stability will have a detrimental effect on the performance of the compartment wall and of the non fire-affected section of the building. This can be avoided by placing a suitable prop in the line of the compartment wall. The prop should be designed to provide stability to the wall in the normal limit state and checked for an axial load from the rafter/ truss in the fire limit state. The connection between the top of the prop and the rafter/ truss in the vertical direction must be such that deflections under normal serviceability conditions are permitted without compressive forces being transmitted into the prop. As an alternative to the prop solution, the cold frame could be designed to have the necessary stability in its own right. This solution would need to consider the stability of the frame subject to dead and imposed loading factored for the fire limit state. The sub-frame to be considered is shown in Figure 3.7. The moment capacity at points of maximum moment should be checked and base fixity will be required to ensure stability of the frame against overturning.

The prop needs only to be designed to carry the fire load from half of the rafter span. The prop needs to be fire-protected, which may be achieved by building the prop into the compartment wall.

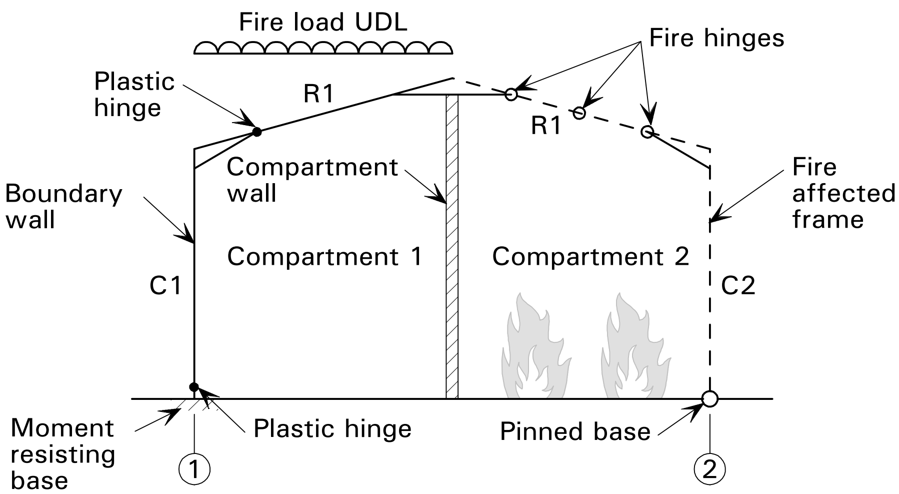


Figure 3.7 *Forces and moments on cold structure when fire affects one compartment*

3.12 Columns not supporting protected areas

Often walls are sufficiently far from site boundaries to require only a proportion of their area to be 'protected area'. It may be convenient or economically desirable to have some columns supporting protected areas and others supporting unprotected areas. In these cases, it is normally reasonable not to fire protect the columns supporting the unprotected areas and not to 'fix' their bases.

A common example of this occurs when a wall is not parallel to the site boundary. The risk of fire spread will be sufficiently reduced if the external wall closest to the site boundary has fire resistance. It would seem reasonable that any column remote from the boundary should not be treated in the same way as a column close to the boundary, which is supporting a protected area.

3.13 Roof venting

In a severe fire, a roof cladding system may be destroyed to such an extent as to enable the fire to vent (i.e. introduce cold air to the fire in reasonably large quantities). The effect of this may be to reduce the maximum temperatures reached within the building. However, depending on the fire load, it may not be sufficient to reduce the temperature of the steel below the critical temperature that would lead to collapse of the rafters. If the fire load is sufficient and collapse occurs, the value of the maximum overturning moment will be unaffected, although the time at which it is attained will depend on the amount of ventilation.

It is important to understand that roof venting is not required if the recommendations contained in this publication are followed. Confusion has sometimes arisen over the need to provide roof vents because an alternative approach, based on a former relaxation given in the *Approved Document* required the roof to contain low melting point roof lights with an area equal to at least 10% of the floor area. This version of the *Approved Document* has now been superseded.

3.14 Frames with only one boundary condition

Buildings are often constructed with a boundary condition on only one side. If a frame is constructed with one column more able to resist overturning than another, it could be argued that the weaker column will collapse first and this could lead to a reduced overturning moment. However, collapse of a weaker column is not inevitable. It is therefore recommended that the overturning moment applied to the columns supporting the protected areas of the external wall be calculated assuming symmetric frame behaviour although only the foundations of the columns on the boundary side need to be designed to resist this moment.

4 LOADING

The recommendations in this publication allow a reduced design load to be used to calculate the overturning moment in fire conditions. This Section explains the basis for that reduction.

4.1 Dead loads

4.1.1 Roof loads

The dead load on the roof structure of a single storey building will be due to the self-weight of the structural members, the purlins and the cladding system. Depending on the type of materials used in the cladding system, a reduced value of dead load may also be assumed. In a fire of sufficient intensity to cause rafter collapse, some of the cladding may be destroyed. The original study[8] of portal frames in fire included an assessment conducted by the Fire Research Station of the extent to which some common roof cladding materials would be destroyed in a fire. Table 2.1, which gives the percentage of commonly used roof cladding materials that would be expected to remain intact at the time of rafter collapse, is based on that assessment. This information can be used to determine an appropriate reduction in the dead weight applied to the rafter during a fire.

4.1.2 Superimposed dead load due to services

In the design of portal frames, it is common to include an allowance for loading due to services. This loading is included to allow for such things as electrical and mechanical services. Values for service loading vary from 0.1 to 0.4 kN/m^2.

In fire, the service loading is assumed to be reduced from the normal level of loading due to some of the loading having never been present in the first instance and secondly due to some of the services becoming detached with the gross rafter deformations which are experienced in fire.

It is proposed that the service loading be reduced to a third of the value used in normal design where this has been imposed as a uniform load. In cases were significant service loads are known to exist, such as those associated with industrial operations, should be specifically considered.

4.2 Imposed loads

Snow forms the principal imposed roof loads for single storey buildings. As the combination of snow and fire are remote BS 5950-8 states that snow loading may be ignored. It is also unlikely that imposed roof loads given in BS 6399-3[15] to allow for access for maintenance will be present in a fire. In most cases therefore, assuming a value of zero for imposed roof load, should be appropriate.

4.3 Wind loads

For normal design, wind loads are taken as a worst case likely to occur over the lifetime of the building, a period usually measured in years. Temporary structures are designed for a reduced value of wind loading depending on the length of time they will be exposed. In the case of fire design, the period under consideration is measured in terms of hours rather than years and therefore a significant reduction in wind loading would seem appropriate, to reflect the reduced probability of fire occurring simultaneously with significant wind loading.

Currently, BS 5950-8 recommends that, for fire design, a load factor of 0.33 be applied to wind loads at the fire limit state. Furthermore, wind loading should only be considered for buildings with a height to eaves of more than 8 m. It is expected that a revised version of BS 5950-8 will recommend that wind loading need not be considered when checking the stability of a boundary wall.

In view of this impending change to BS 5950-8, it is recommended in Section 2.4.2 that wind load be ignored when designing for boundary wall stability in fire, regardless of the height of the wall.

This approach is also consistent with Eurocodes, which currently recommend that wind loading be ignored for the fire limit state.

5 BEHAVIOUR OF SYMMETRICAL PITCHED PORTAL FRAMES

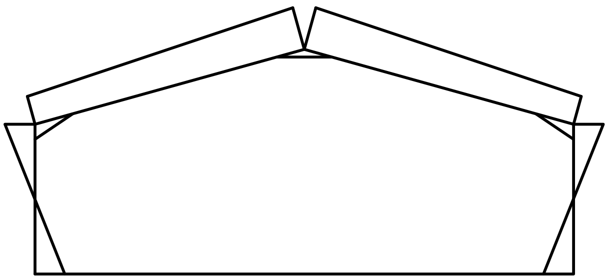
5.1 Single span frames

A study was undertaken in the late 1970's to investigate the behaviour of portal framed single storey buildings in fire. From this study a description of frame behaviour during a fire was developed. The description is equally applicable to frames with fabricated tapered rafters and columns and to frames utilising hot rolled sections.

In the early stages of fire development, the portal rafter begins to heat up and expand, which causes a small outward deflection of the eaves, together with a small upward deflection of the apex.

As the fire continues to burn, the rafter temperature rises and the deflections and moments due to thermal expansion increase. The distribution of the thermally induced bending moments is shown in Figure 5.1. The rise in temperature causes a reduction in the yield strength of the steel and although the loading remains constant, the moment capacity of the rafter will reduce and, if the temperature is sufficiently high, plastic hinges will start to form in the rafter. The term 'fire hinge' is used to distinguish this type of plastic hinge from the plastic hinge which can form at normal temperatures. The moment of resistance of a fire hinge is considerably less than the corresponding value at normal temperature.

The fire hinges tend to form at the ends of the haunches and near to the ridge (Figure 5.2). At this stage, the frame maintains its basic shape.



Note: The apex will deflect upwards and the eaves outwards

Figure 5.1 *Bending moment diagram for uniform temperature rise*

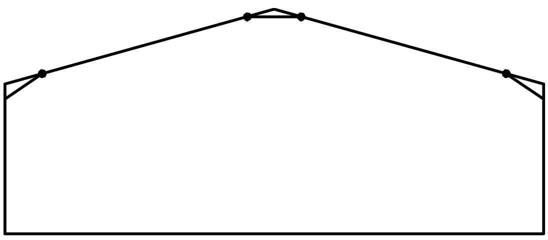


Figure 5.2 *Probable positions of fire hinges in a portal frame*

By this stage, the loading on the frame is its self-weight and purlins but with only a portion of the weight of the cladding and insulation. (The percentage weight of roof cladding likely to remain at the time of rafter collapse is given in Table 2.1.)

With the formation of the hinges, the rafter tends to form into a two or three-pinned arch. Axial thrusts are induced and the rafter, unable to resist, begins to collapse. The base moment induced by thermal expansion quickly reduces to zero and then builds up in the opposite sense (i.e. inwards).

At this stage, the distorted rafter is still able to support itself in a roughly horizontal attitude, as shown in Figure 5.3. The columns are still upright and the purlins afford a degree of stability to the rafter.

The rafter continues to collapse and falls below eaves level but remains reasonably straight between fire hinges. Torsional instability may occur as the purlins lose their strength. The rafter is acting partially as a catenary, creating a tensile load, which pulls on the top of the column with moments acting at the fire hinges at the ends of the haunches and the ridge. Slight elongation of the rafter will have occurred. The columns are still upright and showing little signs of distress.

The rafter continues to collapse as it loses stiffness and the section may rotate so that it sags with its web horizontal. The moment at the end of the haunch still maintains an appreciable value. As the rafter further loses strength, it will continue to sag to below eaves level and begin to pull inwards on the tops of the columns.

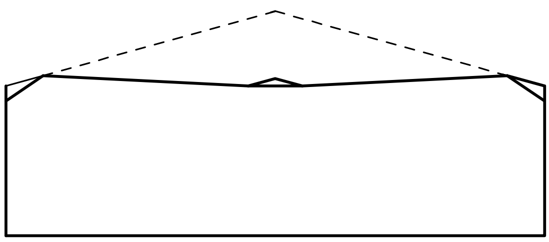


Figure 5.3 *Rafter collapses to eaves level*

A mathematical model of the rafter collapse mechanism at this stage is given in Appendix A. It is the forces and moments at this stage that are used to determine the overturning moment.

Simplified calculation method for symmetrically pitched portal frames

The full calculation method applied to the overturning moment is somewhat tedious to use manually. A simplified method of calculation was devised which was shown to be sufficiently accurate. A study of a large number of frames showed that, subject to certain conditions, the simplified method gave results within 10% of the full method, usually within 5%. For frames in which the span divided by the height to eaves is less than 1.6, the simplified method gave very conservative values.

In order to reduce the conservatism of the simplified method to the frames with span/height ratios of less than 1.6, the assumptions initially made in the derivation of the simple method were reviewed and a second set of parameters produced. As a result, an improved set of parameters has been produced for span: height ratios between 1.1 and 2.1. The existing set of geometric parameters are now applied to frames with span: height ratios greater than 2.0.

The simplified method, with parameters for ratios above and below 2.0 is given in Section 2.5.1 and the derivation of the method is described in Appendix B.

5.2 Multi-bay frames

In the original study of portal frames in fire, seven of the eight cases studied were buildings with multi-bay portal frames, but collapse of an internal column only occurred in one case, as shown in Figure 5.4.

In this example, the column buckled about its weak axis and the head of the column sagged by approximately 30% of the height of the column. As the other steelwork, such as valley beams and purlins, effectively restrained the portal frames, (preventing significant out-of-plane deformation), the column head remained above the base during collapse.

Figure 5.4 *Collapse of internal column*

Other similar structures that were included in the original study did not suffer internal column collapse even when subjected to severe fires, resulting in severe rafter deformation.

In the event of a large fire of sufficient intensity to cause collapse of the internal column as well as the rafter, the equations for calculating the overturning moments for single bay frames can be modified to allow for the affect of the partial collapse of the internal column. The collapse of the internal column affects frames with low span to height ratios, typically less than 2.5. For frames with span to height ratios less than the lower limit given in Table 2.3, the internal column should be fire-protected.

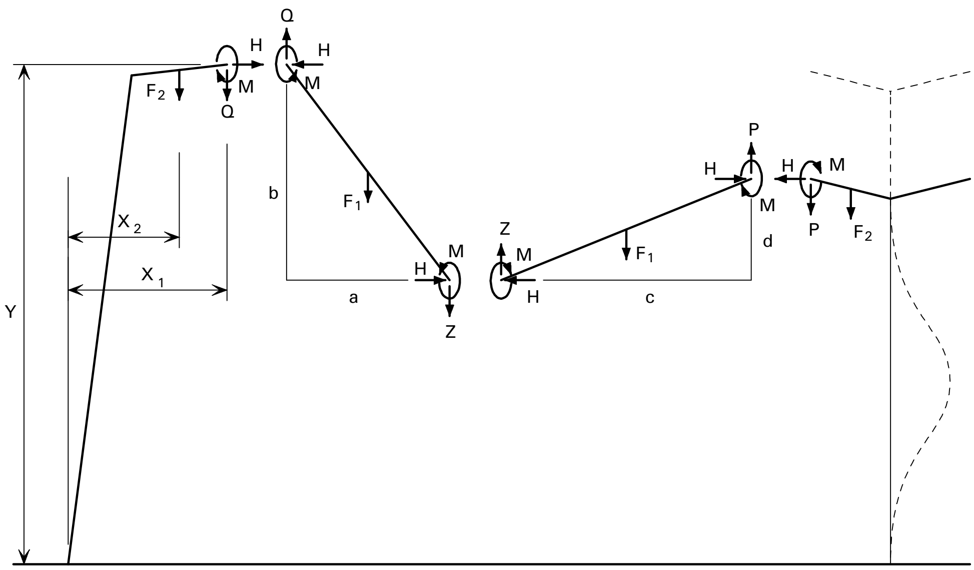


Figure 5.5 *Forces and moments caused by the collapsing rafter.*

The model for calculating the overturning moment for a multi-bay frame, including allowance for sagging of the penultimate column, is as follows. Considering the end bay of a frame with the dimensions, forces and moments as shown in Figure 5.5, for rafter equilibrium:

$$Q + P = 2F_1$$

$$Hb + \frac{F_1\, a}{2} + 2M = Qa$$

$$Hd + \frac{F_1\, c}{2} + 2M = Pc$$

$$P = \frac{1}{c}\left(Hd + \frac{F_1 c}{2} + 2M \right)$$

$$Q = 2F_1 - P$$

$$H = \frac{F_1 c - 2M\left(1 + c/a\right)}{d + bc/a}$$

$$OTM = HY + F_1\, X_1 + F_2\, X_2 + M$$

Where:

H	is the horizontal force in the rafter
Y	is the height to the end of the haunch
F_1	is the vertical load on the rafter
F_2	is the vertical load on the haunch
X_1	is the horizontal distance from the column base to the end of the haunch
X_2	is the horizontal distance from the column base to the line of action of force F_2
M	is the moment capacity of the rafter in fire.

The external column is assumed to lean inwards by 1° but the internal column is assumed to remain upright as the rafter collapses. The rafter is assumed to elongate by 2%, as for the case with single bay frames.

Multiplication factor, K

In order to determine the affect of the partial collapse of the internal column on the magnitude of the overturning moment on the external columns of a multi-bay frame, a parametric study was conducted. The study included frames with span to height ratios of 1.4 to 2.5 and roof pitches from 3° to 15°.

This study showed that the increase in overturning moment was more pronounced in low-pitched portal frames and in frames where the span divided by the height to eaves is small. Therefore, it is recommended that for multi-bay frames the overturning moment be calculated by applying a multiplication factor to the moment calculated using the simple model for symmetrical single span pitched portal frames.

If the collapse of the internal column results in an increase of overturning moment up to 10%, a multiplication factor of 1.0 may be used. It was considered that, as the simplified method is conservative, 10% increase in moment could be allowed before an increase in the multiplication factor is required. If the increase in overturning moment due to the collapse of the internal column is between 10% and 30% a factor of 1.3 should be adopted. In cases where the increase in moment due to collapse of the internal column is greater than 30%, fire protecting the column is recommended.

6 BEHAVIOUR OF TRUSSES AND LATTICE RAFTERS

The consideration of trusses and lattice rafters was omitted from the original study, as building regulations at that time did not require trusses to be fire-protected because of the existence of a boundary condition. However, as the regulations move away from an entirely prescriptive approach, questions are often asked regarding the perceived anomaly between the consideration given to the stability of a boundary wall of a portal framed single story building and the boundary wall of a single storey building with a trussed roof. While no evidence has been brought forward to show that a problem exists, it now seems sensible, given the wide acceptance of the SCI method for portal frames in boundary conditions[10], to treat all single storey buildings in a similar manner.

6.1 Behaviour of trusses in fire

As trusses are constructed from relatively slender struts and ties, they are particularly vulnerable in a fire because of the rapid rise in temperature of these members. The compression members would be expected to fail in the early stages of a fire, therefore any bending resistance will be provided only by the top and bottom chords of the truss. In fire, the main mechanism for load transfer will be tensile action of the top and bottom chord of the truss.

Adopting an approach similar to that used for portal frames, assuming the worst case of an inverted roof structure, the horizontal pull on the column tops can be calculated by considering the roof to behave as a catenary as shown in Figure 6.1.

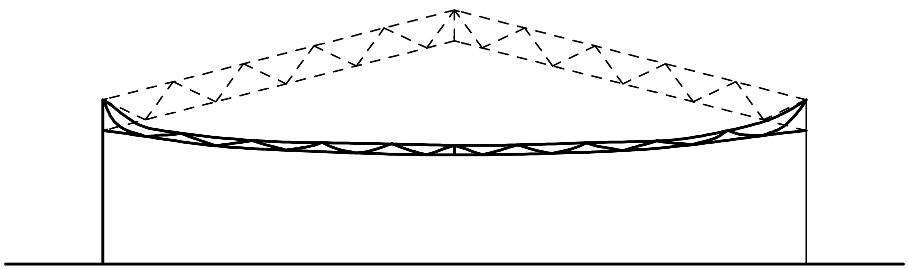


Figure 6.1 *Collapse mechanism of a typical lattice rafter during a fire.*

6.1.1 Mathematical model

Trusses can be modelled using the same model as is used for symmetrical pitched portal frames (Appendix A). However, because of the mode of failure of trusses, the residual moments in the rafter are assumed to be zero and the haunch length is also assumed to be zero.

As this method ignores bending of the chords of the truss, the magnitude of the overturning moment will be expected to be higher than the overturning moment for portal frames.

A study was conducted to determine the magnitude of the difference between overturning moments for trusses and portal frames. The study carried out considered a truss and a portal frame with spans of 15, 20, 25 and 30 m, height to eaves of 6, 7, 8, 9 and 10 m and roof pitches of 0°, 3° and 6°.

As expected, the overturning moments calculated for the truss using the simplified method were 1.5 to 2.0 times those calculated for the portal frame. Table 6.1 shows the results of this study for frames with a height to eaves of 6 m, a frame spacing of 5 m and a fire loading of 0.3 kN/m^2.

Table 6.1 *Comparison of overturning moments calculated for trusses and portal frames.*

Roof pitch	Frame span *L* (m)	Mid Span Sag *Y* (m)	Truss: Horizontal reaction (kN)	Truss: *OTM* (kNm)	Portal frame: Horizontal reaction (kN)	Portal frame: *OTM* (kNm)	Ratio of *OTM* truss/portal
0°	15	1.96	23	135	8	72	1.9
	20	2.47	30	180	12	110	1.6
	25	2.99	38	225	15	151	1.5
	30	3.50	45	270	18	200	1.4
3°	15	2.00	22	134	8	72	1.9
	20	2.53	30	179	11	109	1.6
	25	3.06	37	225	14	151	1.5
	30	3.59	44	271	18	199	1.4
6°	15	2.12	21	128	8	70	1.8
	20	2.70	28	172	11	106	1.6
	25	3.27	35	216	13	147	1.5
	30	3.85	42	261	17	195	1.3

6.1.2 Application of the model

Trussed roofs can be framed in a variety of ways using basic truss types. Some of the common types of truss are shown in Figure 6.2 to Figure 6.4. The calculation of the overturning moment should be based on the minimum pitch of the top and bottom chord.

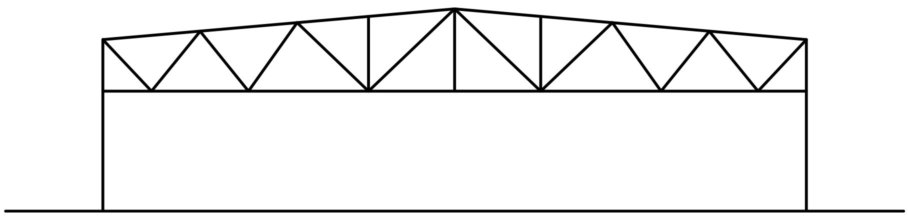


Figure 6.2 *Typical trussed roof with moment connections to columns.*

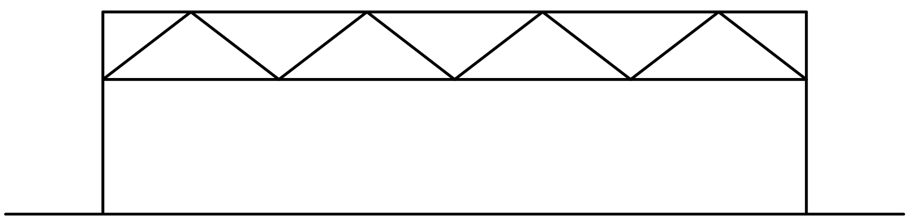


Figure 6.3 *Roof with warren truss rafters*

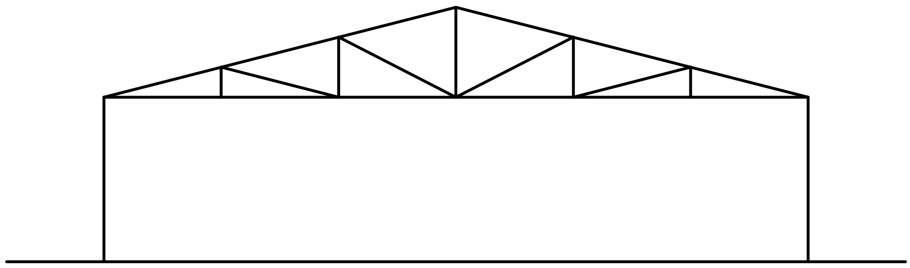


Figure 6.4 *Typical pitched truss roof with pinned connections to columns.*

7 BEHAVIOUR OF LEAN-TO STRUCTURES

7.1 General

Single storey buildings are often constructed with a portal frame and a lean-to structure that relies in part on the main frame for stability. The behaviour of such structures has been examined using the simple frame shown in Figure 7.1. The structure considered consists of a symmetrical pitched portal frame with a mono-pitched lean-to structure on one side. The mono pitched rafter R2 is connected to the columns C2 and C3 with pinned or moment connections.

A number of possible boundary conditions were considered, and for each, a method of maintaining the stability of the columns supporting boundary walls was determined. The structure was also considered with a compartment wall dividing the building into two compartments and the stability of the compartment was also addressed.

The design requirements are explained in terms of eight design scenarios, covering a range of boundary conditions and compartment wall positions.

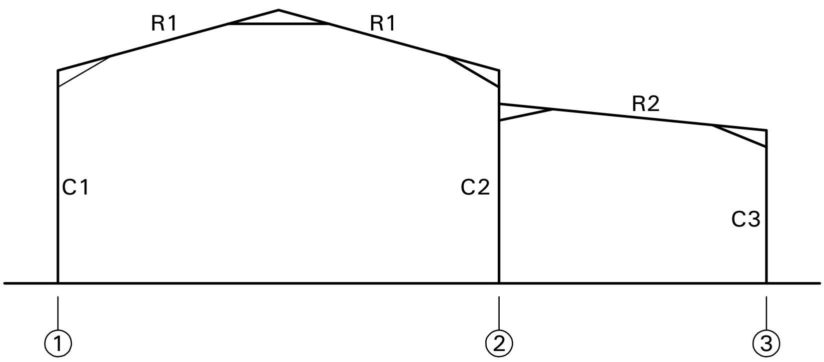


Figure 7.1 *Basic portal frame with lean-to section*

7.2 Design scenarios

7.2.1 Case 1: Building with no compartment walls

Case 1a: Boundary condition on gridline 1

It is assumed that both rafters are affected by fire and collapse as a result, as shown in Figure 7.2.

Column C1 will be subject to a horizontal force due to the collapse of rafter R1. As this column is supporting a boundary wall, it must remain stable and will require a moment resisting base to prevent collapse.

The collapse of rafter R2 will not adversely affect Column C1. Columns C2 and C3 will not require fire-resisting bases or fire protection.

Summary of design requirements

Column C1: Fire protect the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Column C2: No special considerations for fire

Column C3: No special considerations for fire

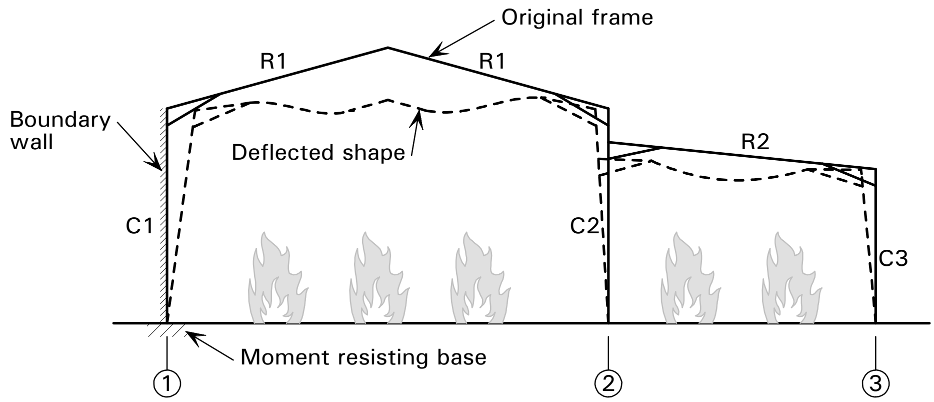


Figure 7.2 *Case 1a: Collapse mechanism when both rafters are affected by fire, boundary on gridline 1.*

Case 1b: Boundary condition on gridlines 2 and 3

Again it is assumed that the whole building can be affected by fire simultaneously and that both rafter R1 and R2 will collapse as a result, as shown in Figure 7.3. Columns C2 and C3 will require fire protection to the underside of the haunch. The bases of columns C2 and C3 will also require moment resistance in order to resist the overturning moments resulting from the horizontal forces on the columns generated by rafter collapse.

Depending on the relative spans of rafters R1 and R2, allowance will have to be made for additional forces being transmitted to column C3 as a result of the collapse of rafter R1.

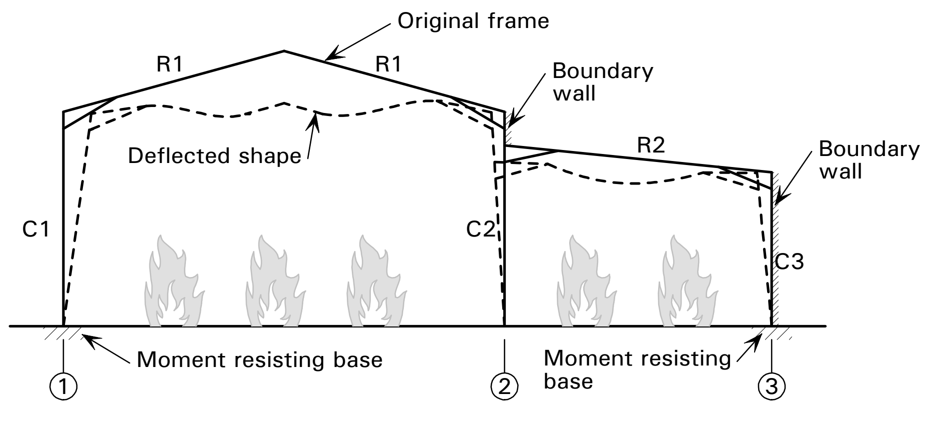


Figure 7.3 *Case 1b: Collapse mechanism when both rafters are affected by fire, boundary condition on gridlines 2 and 3.*

Summary of design requirements

Column C1: No special considerations for fire

Column C2: Fire protect the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Column C3: Fire protect the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Case 1c: Boundary condition on gridline 3

This case is similar to Case 1b, except that the boundary condition exists on gridline 3 only. Column C3 should be fire-protected and have a moment resisting base. The horizontal forces produced by the collapse of rafters R1 and R2 should be considered and the higher of the two values should be used in the calculation of the overturning moment on the base of C3.

Summary of design requirements

Column C1: No special considerations for fire

Column C2: No special considerations for fire

Column C3: Fire protect the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

7.2.2 Case 2: Compartment wall on gridline 2

In this case, the building is divided into two compartments by a compartment wall on gridline 2. Fire is only considered to affect one compartment at a time, as shown in Figure 7.4 and Figure 7.5.

Case 2a: Boundary condition on gridline 1

Fire in compartment 1

In the case of fire affecting compartment 1, as shown in Figure 7.4, and causing the collapse of rafter R1, the base of column C1 will have to be designed for the horizontal force applied to the top of the column from the collapsing rafter. Column C1 will also require fire protection.

Columns C2 and C3 will not require fire protection to satisfy the boundary condition. However, as column C2 supports a compartment wall it will need fire protection to satisfy the insulation and stability requirements of the wall.

Fire in compartment 2

In the case of fire affecting compartment 2 (Figure 7.5), the collapse of rafter R2 will have very little effect on the behaviour of column C1. As the horizontal force from rafter R2 on column C2 will be resisted by the cold portal frame C1-R1-C2, it will not have an adverse affect on the stability of column C1.

Summary of design requirements

Column C1: Fire Protect the column to achieve the same fire resistance as the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Column C2: No special considerations are required for the fire boundary condition. Fire protection will be required if the column supports the compartment wall.

Column C3: No special considerations for the fire boundary condition

When fire occurs in compartment 1, it would be unacceptable if the collapse of rafter R1 caused the collapse of the cold structure or allowed the fire to spread to compartment 2. Therefore, the stability of the cold frame and the compartment wall should also be considered. Guidance on how to achieve this is given in Section 7.3.3.

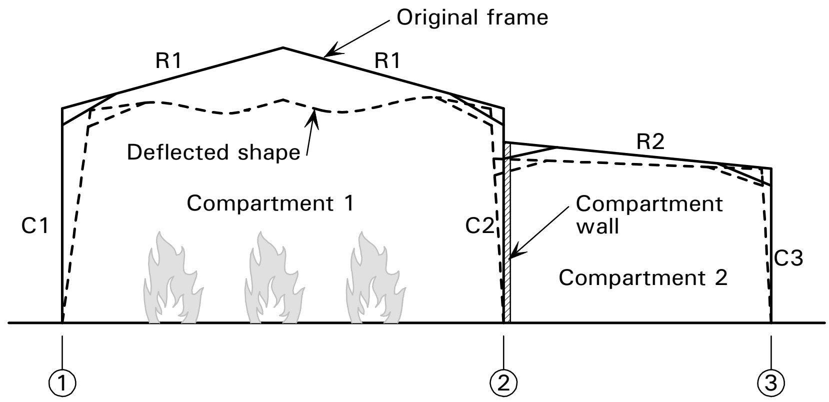


Figure 7.4 *Case 2: Collapse mechanism for frame with compartment wall on gridline 2, fire in compartment 1.*

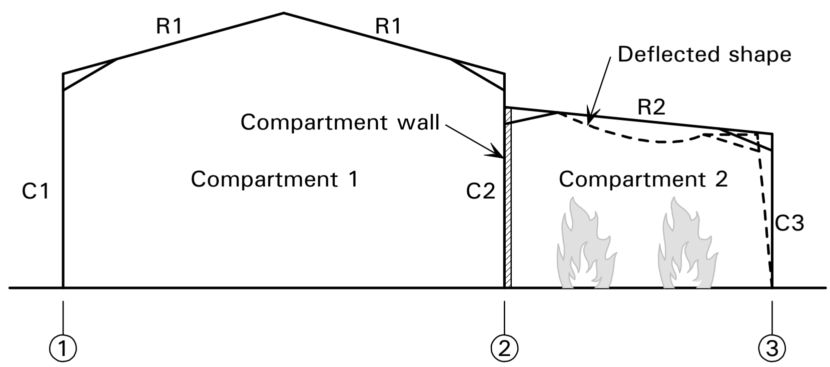


Figure 7.5 *Case 2: Collapse mechanism for frame with a compartment wall on gridline 2, fire in compartment 2.*

Case 2b: Boundary condition on gridlines 2 and 3.

Depending on the span of rafter R2, the external portion of the wall on gridline 2, above the level of rafter R2, may also be in a boundary condition.

Fire in compartment 1

If compartment one is affected by fire (Figure 7.4), causing rafter R1 to collapse, column C2 will need to be fire-protected and to have a moment-resisting base capable of resisting the horizontal force H_{R1} resulting from the collapse of rafter R1.

Alternatively, the cold frame C2-R2-C3 may be designed to resist the horizontal force H_{R1}, as shown in Figure 7.6. Depending on the magnitude of the horizontal force H_{R1}, this will reduce or eliminate the need for base fixity on column C2.

The fire resistance and stability of column C1 is not a concern in this case, because it is not on a boundary. Also, where column C2 is designed to resist the horizontal reaction from the collapse of rafter R1, column C3 will not be affected.

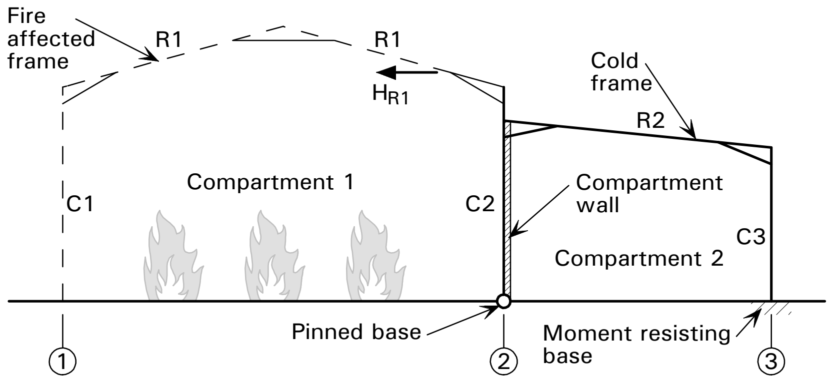


Figure 7.6 *Cold frame resisting horizontal force from rafter R1*

Fire in compartment 2

The situation is shown in Figure 7.5, where rafter R2 is affected by the fire. Column C3 will need fire protection and a moment resisting base capable of resisting the moment from the collapse of rafter R2.

For this case, column C2 will not need a moment resisting base as it is stabilised by the cold structure C1-R1-C2.

Summary of design requirements

Column C1: No special considerations for the fire boundary condition.

Column C2: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column. Alternatively, the cold steelwork on either side of the boundary wall could be designed to resist the horizontal force from the collapsing rafters, thus ensuring the stability of column C2 without the need for a moment resisting base.

Column C3: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Case 2c: Boundary Condition on gridline 3

Fire in compartment 1

If the fire occurs in compartment 1, as shown in Figure 7.4, rafter R1 will be heated. The collapse of this rafter will apply a horizontal force, H_{R1}, to column C2. To maintain the stability of the compartment wall on gridline 2 and the boundary wall on gridline 3, the base on column C2 can be designed to resist the moment, which results from this force. Alternatively, the stability of the cold frame C2-R2-C3 may be checked under the horizontal reaction H_{R1} caused by rafter collapse.

Fire in compartment 2

If the fire occurs in compartment 2, as shown in Figure 7.5, the rafter R2 will be heated and column C3 needs to be designed to resist the forces resulting from the collapse of rafter R2. Column C3 will be fire-protected to the same standard as the wall it supports and will have a moment resisting base. The compartment wall on gridline 2 will remain stable during the fire as the cold structure C1-R1-C2 will provide stability. Column C2 will be fire-protected to satisfy the requirements of the compartment wall.

Summary of design requirements

Column C1: No special considerations for the fire boundary condition.

Column C2: No special considerations for the fire boundary condition if frame C2-R2-C3 is designed to resist the horizontal force from the collapse of rafter R1. May require insulation to satisfy insulation requirements for the compartment wall.

Column C3: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

7.2.3 Case 3: Compartment wall at the mid-span position of rafter R1.

Case 3a: Boundary Condition on Gridline 1.

Fire in compartment 1

If the fire affects compartment 1, as shown in Figure 7.7, the fire exposed section of the rafter R1 will collapse. This will exert a horizontal force on the top of column C1, which can be resisted by fixing the base of column C1.

With the removal of portal action, the remaining cold frame will have to resist a vertical and a horizontal reaction from the heated section of the rafter. The stability of the cold frame must be maintained to prevent disproportionate collapse and to enable the integrity of the compartment wall to be maintained.

The vertical reaction can be supported by a fire-protected prop in the line of the compartment wall. The use of a prop will eliminate the possibility of the cold section of the portal rafter collapsing due to the formation of a plastic hinge in the rafter or column C2 at eaves level. Alternatively, the moment capacity of the rafter could be checked to ensure that this mode of failure will not occur.

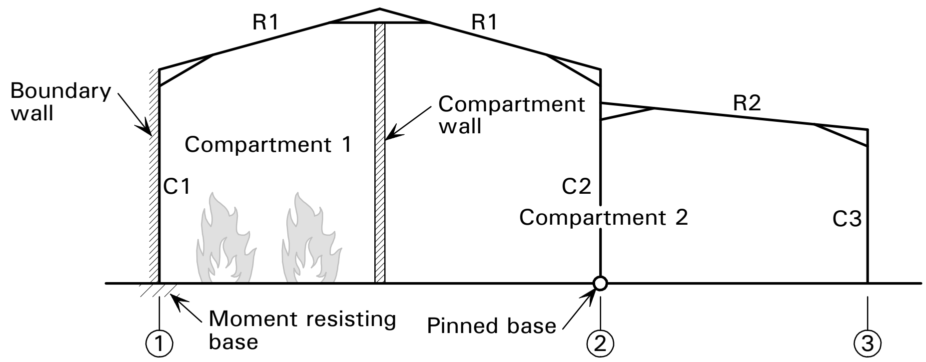


Figure 7.7 *Case 3a: Boundary condition on gridline 1, fire in compartment 1.*

Fire in compartment 2

If fire affects compartment 2, as shown in Figure 7.8, the fire-affected section of rafter R1 will collapse. The same is true for rafter R2. The collapse of rafter R1 will exert an additional horizontal and vertical force on the cold frame, which must be resisted by the base of column C1. To ensure that the deflection of the cold rafter does not affect the integrity of the compartment wall and to support the vertical reaction from the fire-affected section of the rafter R1, a prop could be provided in the plane of the wall. This will prevent the formation of a plastic hinge in the cold frame at eaves level or at the base of the column, which would lead to disproportionate collapse and failure of the compartment and boundary walls.

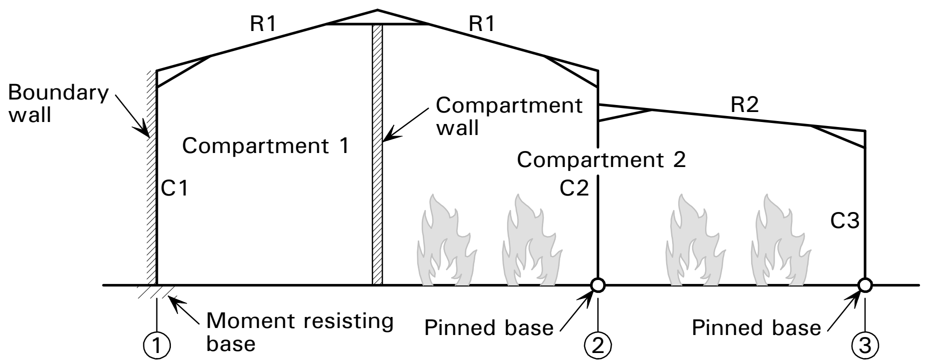


Figure 7.8 *Case 3a: Boundary condition on gridline 1, fire in compartment 2.*

Summary of design requirements

Column C1: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Column C2: No special considerations for the fire boundary condition.

Column C3: No special considerations for the fire boundary condition.

Providing a prop to the main portal rafter in the plane of the compartment wall will reduce the design moment on the base of column C1 and will also prevent the deflection of the rafter disrupting the wall. The prop will only need to be designed to resist the fire loading.

Case 3b: Boundary condition on gridline 3

Fire in compartment 1

If compartment 1 is affected by fire, as shown in Figure 7.9, the effect of the collapse of the heated portion of R1 on the stability of the cold structure in compartment 2 and on the compartment wall needs to be considered. The collapse of the fire-affected rafter will result in a horizontal and vertical reaction occurring at the ridge of rafter R1. The collapse of the cold rafter R1 would be unacceptable in this case, as it would disrupt the compartment wall. The easiest solution is to provide a fire-protected prop within the compartment wall. The prop should be designed for the vertical loading from the fire-affected part of R1 and for the vertical loading due to propping of the cold frame. Alternatively, the rafter could be checked against the formation of a plastic hinge under both the horizontal and vertical loads from the fire-affected portion of R1.

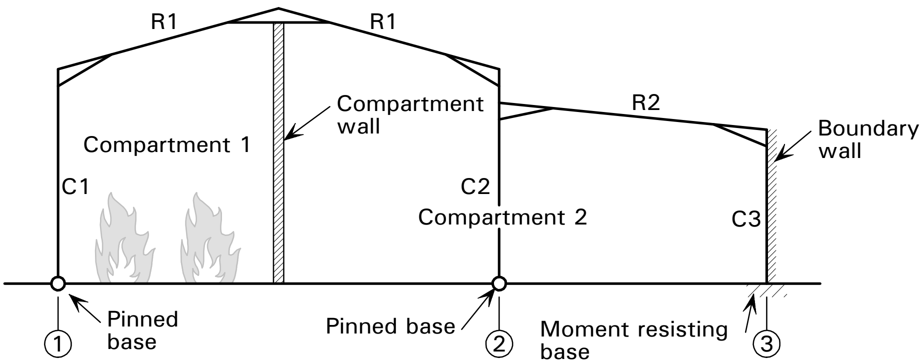


Figure 7.9 *Case 3b: Boundary condition on gridline 3, fire affects compartment 1.*

Fire in compartment 2

If compartment two is affected by fire, as shown in Figure 7.10, the base of column C3 must be designed to resist the total horizontal force resulting from the collapse of rafters R1 and R2.

The loss of portal action in rafter R1 will have a significant effect on the steelwork in compartment one. Vertical and horizontal reactions will arise on rafter R1 at the ridge position. In order to maintain the stability of the remaining cold frame and the compartment wall, the remaining structure must be designed to resist these forces as well as the gravity loading applied to the cold rafter. The easiest solution is to provide a prop in the line of the compartment wall. The prop should be designed for the loads from the fire-affected structure and the load on the cold portion of the frame. Alternatively, the base of column C1 could be designed to resist the applied moments, provided that rafter R1 does not form a plastic hinge prematurely.

Summary of design requirements

Column C1: No special considerations for the fire boundary condition.

Column C2: No special considerations for the fire boundary condition.

Column C3: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

In order to prevent a fire in either compartment affecting the stability of the cold portion of the frame, a prop to the main portal rafter in the plane of the compartment wall should be provided; this would also prevent the deflection of the rafter disrupting the wall. The prop will need to be designed to resist the loading from the fire-affected and cold parts of the frame.

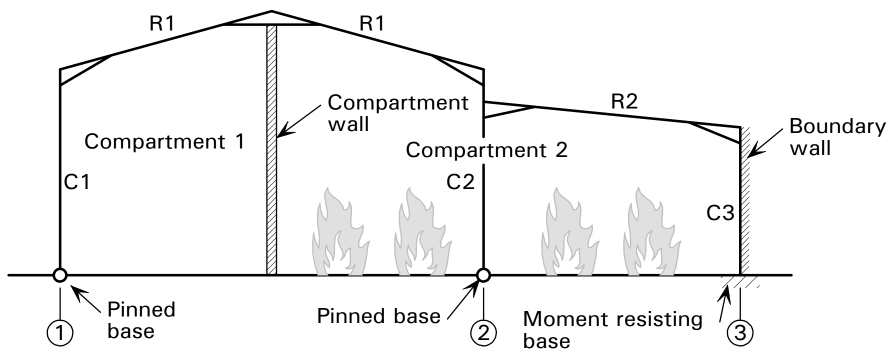


Figure 7.10 *Case 3b: Boundary condition on gridline 3, fire in compartment 2.*

7.2.4 Case 4: Compartment wall beyond the mid span of rafter R1

Case 4a: Boundary Condition on gridline 1

Fire in compartment 1

If the fire occurs in compartment 1, as shown in Figure 7.11, the fire-exposed section of rafter R1 will collapse. This will exert a horizontal pull on the top of column C1, which can be resisted by fixing the base of C1.

Horizontal and vertical forces will also be exerted on the cold frame. To prevent disruption of the compartment wall, allowance must be made for an increased vertical reaction at this point. A prop may be provided in the plane of the compartment wall, as in Case 3. Alternatively, the cold section of rafter R1 should be checked for the increased moment due to the reactions.

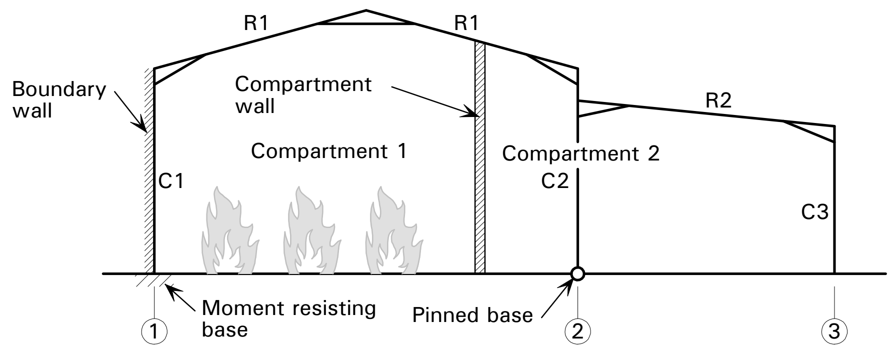


Figure 7.11 *Case 4a: Compartment wall located beyond the mid span location, fire affects compartment 1.*

Fire in compartment 2

If the fire affects compartment 2, as shown in Figure 7.12, the collapse of rafter R2 will exert a horizontal force on the main frame. The loss of portal action in the main frame, due to the heating of part of rafter R1, will also result in increased moments in the cold frame. It is unlikely that the increases could be resisted by the base fixity on column C1 or the rafter R1; it will be necessary to provide a prop in the plane of the compartment wall. Alternatively, if the base of column C1 can be designed to resist the increased moment, then rafter R1 must also be checked for the formation of a plastic hinge at the eaves.

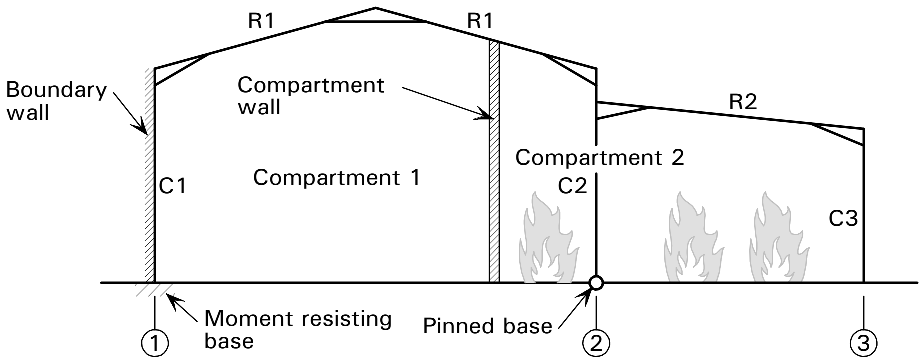


Figure 7.12 *Case 4a: Compartment wall located beyond the mid span location, fire affects compartment 2*

Summary of design requirements

Column C1: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

Column C2: No special considerations for the fire boundary condition.

Column C3: No special considerations for the fire boundary condition.

Providing a prop to the main portal rafter in the plane of the compartment wall will reduce the design moment on the base of column C1 and will also prevent the deflection of the rafter disrupting the wall.

Case 4b: Boundary condition on gridline 3

Fire in compartment 1

For a fire in compartment 1, as shown in Figure 7.13, the stability of the cold part of the frame must be checked to ensure it can resist the horizontal reaction from the collapse of the heated section of rafter R1.

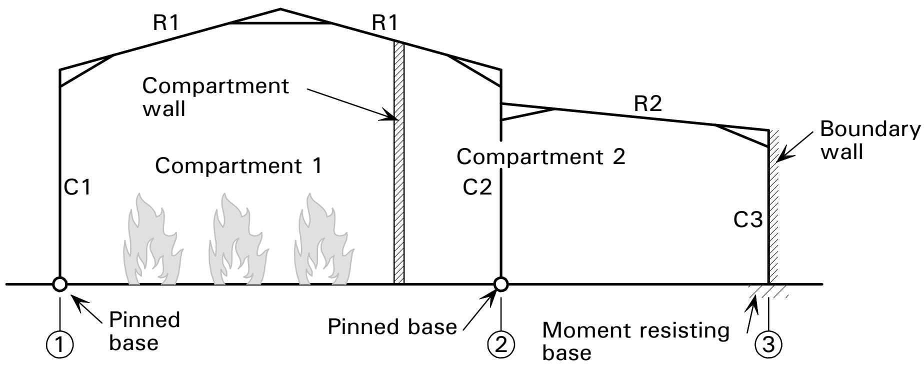


Figure 7.13 *Case 4b:Boundary condition on gridline 3, fire in compartment 1.*

Fire in compartment 2

For a fire in compartment 2, as shown in Figure 7.14, the base of column C3 will need to be designed to resist the horizontal force from the collapse of the fire-affected rafters.

The loss of portal action in rafter R1 will affect the stability of the cold frame contained within compartment one. To prevent collapse of the frame or disruption of the compartment wall, the stability of the frame must be considered. Additional horizontal and vertical reactions will occur, due to the collapse of the heated section of rafter. The cold frame needs to be designed to resist these forces as well as the gravity load present in fire on the cold rafter.

As before, the base of column C1 can be designed to resist the resulting moments or a prop can be provided in the line of the compartment wall. The risk is that the formation of a fire hinge in the heat-affected section of rafter R1 will in turn lead to the formation of a plastic hinge in the cold section of rafter R1.

Summary of design requirements

Column C1: No special considerations for the fire boundary condition.

Column C2: No special considerations for the fire boundary condition.

Column C3: Apply fire protection to the column to achieve the same fire resistance as achieved by the boundary wall it supports. Design the base of the column to resist the overturning moment resulting from the collapse of the rafter supported by the column.

In order to prevent a fire in compartment 2 affecting the stability of the cold frame, a prop to the main portal rafter in the plane of the compartment wall could be provided; this will also prevent the deflection of the rafter disrupting the wall. The prop will only need to be designed to resist the fire loading.

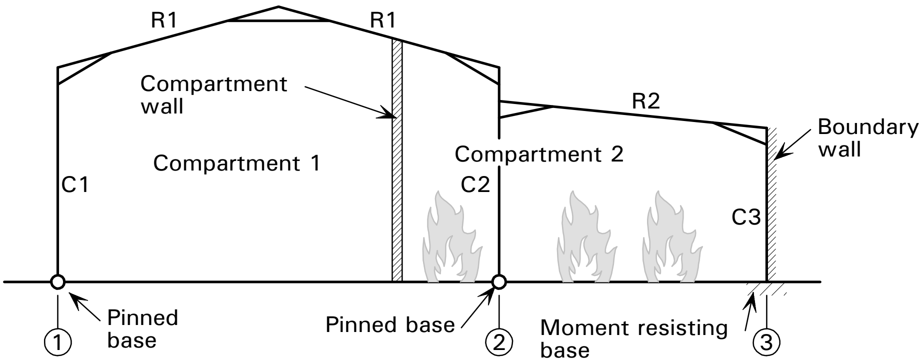


Figure 7.14 *Case 4b: Boundary condition on gridline 3, fire in compartment 2.*

7.3 Compartment walls

This Section gives some more detailed guidance for the above design scenarios where either the main portal rafter or the rafter of the lean-to structure span over a compartment wall.

7.3.1 Calculation of horizontal reaction from rafter collapse

Where the whole of the rafter is heated, the horizontal reaction due to collapse of the rafter is calculated using the methods given earlier in Section 2 for symmetrically pitched and mono pitched rafters.

If only part of the rafter is heated, because of the presence of a compartment wall crossing the rafter at some point on its span, the calculation methods need to be adapted to suit the particular structural arrangement. If the compartment wall occurs at mid span of a symmetrical pitched rafter, the horizontal reaction caused by the collapse of the portion of rafter on either side of the wall should be calculated using the rules for mono pitched rafters.

The same approach is to be adopted for a compartment wall positioned between mid span and eaves. However, for the section of rafter that includes the apex, the horizontal force should be calculated based on a monopitch frame that has the same rafter length as the length of the pitched rafter, between the eaves and the top of the compartment wall as shown in Figure 7.15.

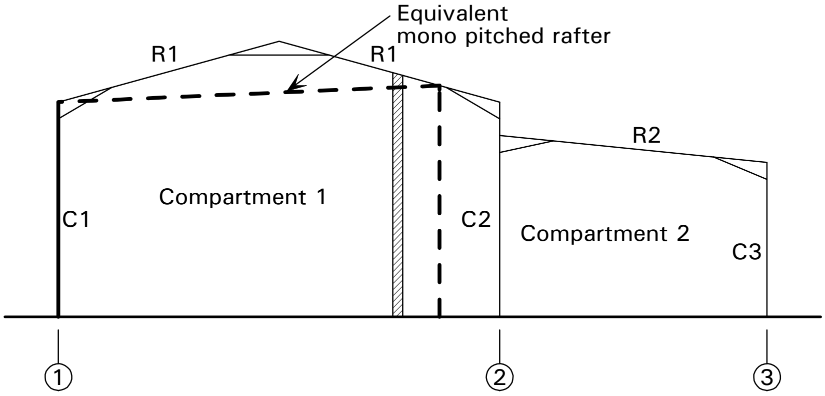


Figure 7.15 *Equivalent mono pitched rafter.*

7.3.2 Design of rafter prop

It is intended that the rafter prop is only effective in the fire condition. For initial sizing purposes, the rafter prop should be designed for a gravity load equal to the half fire design the rafter. When checking the stability of a cold frame as described in Section 7.3.3, the force in the prop may increase, depending on whether or not the column bases are designed to be moment resisting for the fire limit state.

7.3.3 Stability of the cold structure

The loss of continuity in all or part of a main portal rafter when the building is divided into fire compartments will result in an additional horizontal and vertical reaction on the cold part of the structure. The stability of the cold structure under the action of these forces must be considered. The three main concerns are: that any loss of stability will cause disproportionate collapse beyond the fire compartment; that loss of integrity in the compartment wall will allow the fire to spread within the building; and collapse of the boundary wall will allow fire to spread externally.

In most cases providing a fire-protected prop in the line of the compartment wall is the most practical way of supporting the cold rafter.

For the structure shown in Figure 7.16, the lean-to structure will in most cases be sufficient to resist the horizontal reaction from the collapsing rafter if the rafter to column connections are moment resisting. If the rafter is simply supported, the cold structure may be too flexible and its deflection under the action of H_{R1} may cause instability of the compartment wall. Therefore, in this case a moment resisting base should be provided on column C2.

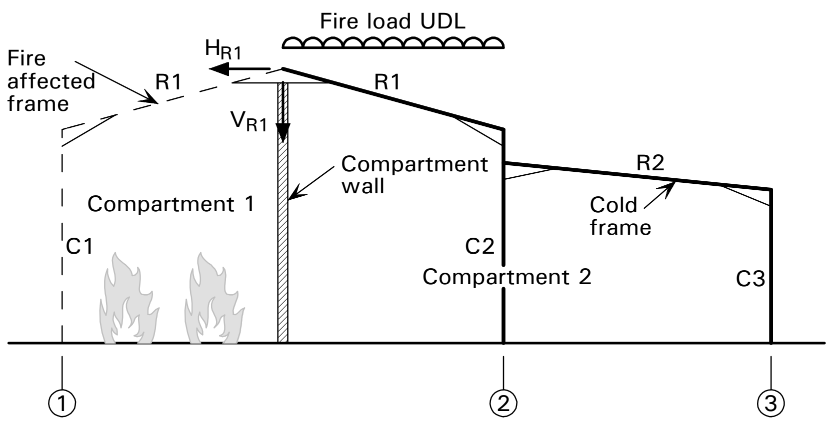


Figure 7.16 *Horizontal and vertical actions on the cold lean-to rafter due to loss of continuity in the portal rafter*

Figure 7.17 shows the same basic structure but in this case with compartment 2 affected by fire. A fire-protected prop is provided in the line of the compartment wall to support the cold rafter. The effect of the sway deflection of the cold structure due to horizontal loads on the stability of the compartment wall should also be checked. If required, the moment resistance of the base of column C1 should be increased. The moment capacity of the portal column and rafter should be checked, the horizontal and vertical deflection of the frame at the top of the wall should be checked and the capacity of the prop should be checked.

Alternatively, the cold frame could be designed as free standing. In this case, the moment resisting base on column C1 will have to be designed for the moments generated by the design load at the fire limit state, the vertical reaction from the fire-affected section of the rafter V_{R1} and the horizontal reaction from the fire-affected rafter H_{R1}. In addition to checking the capacity of the base and the moment capacity of the rafter and column sections, the deflection at the top of the compartment wall should also be checked, to ensure that its integrity is not jeopardised.

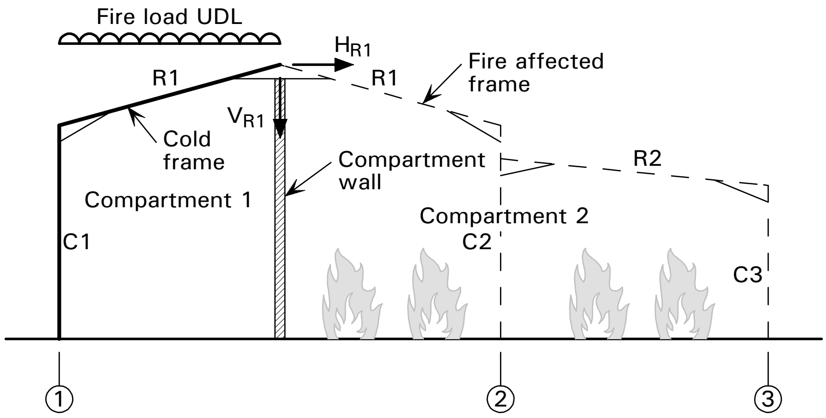


Figure 7.17 *Stability of cold frame when fire occurs in compartment*

8 BEHAVIOUR OF TWO STOREY SECTIONS

Sections of single storey buildings are often partitioned off for use as office accommodation. These sections are often two storey and require some special considerations in relation to fire safety.

The first floor steelwork will require fire resistance, in accordance with the building regulations. The compartment wall between the office section and the rest of the building will also require fire resistance and the possibility of rafter collapse adversely affecting the stability/integrity of the compartment wall should be considered. Where a boundary condition exists, the stability of boundary walls must be maintained.

A number of scenarios are possible, depending on the boundary conditions.

8.1 Design scenarios

8.1.1 Case 1: Compartment wall perpendicular to frames

Case 1a: Boundary condition on gridline 1

The location of the compartment wall and boundary wall is shown in Figure 8.1. The external wall on gridline 1 is in a boundary condition. The main considerations are the stability of the boundary walls, the maintenance of the compartmentation and avoidance of the disproportionate collapse of parts of the structure not affected by fire.

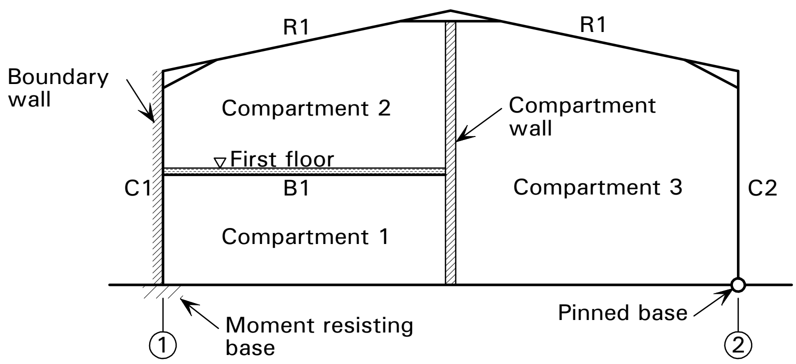


Figure 8.1 *Case 1a: Boundary condition of gridline 1*

If a fire occurs in compartment 1, it will not seriously affect the stability of the building. The main cause of concern to the designer should be a fire in compartment 2 or compartment 3, which will affect the main portal rafter.

The building should be designed so that a fire in compartment 3 will not cause fire spread to compartments 1 or 2, cause the two storey section of the building to collapse, or cause the boundary wall to collapse. The collapse of column C2 and the portion of the rafter in compartment 3 is not a significant problem, provided that the stability of the cold structure is not jeopardised.

The building should also be designed so that a fire in compartment 2 should not cause fire spread to compartment 3 or cause the cold frame associated with compartment 3 to lose its stability.

When fire occurs in compartments 2 or 3, a loss of continuity in the rafter will cause significant deflections to occur at the intersection of the rafter and the compartment wall. Allowance must be made to prevent these deflections from affecting the stability or the integrity of the compartment wall and thus allowing fire spread to the two storey office section. It is recommended that a prop be included in the plane of the compartment wall that will support the rafter in fire conditions.

The cold frame must remain stable under the action of the horizontal and vertical reactions from the collapsing rafter, as shown in Figure 8.2.

For the single storey section of the frame, due to the loss of continuity in the rafter it will be necessary to provide an additional vertical support to the cold part of the rafter. This can be achieved by providing a fire-protected prop to the rafter in the plane of the compartment wall. The horizontal reaction due to the collapse of the fire-affected section of rafter is small and can be resisted by a combination of the cold compartment 3 and compartment 1 structures.

The compartment 1 structure can stabilize the two storey section. The end connections to beam B1, although probably designed as simple connections, would, in fire, normally be sufficiently strong to give adequate sway stability.

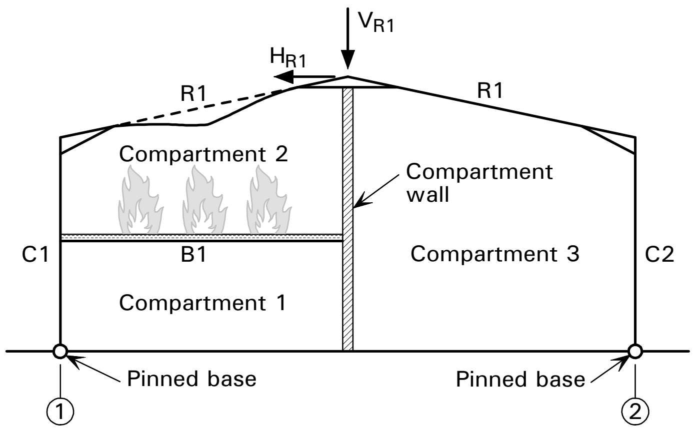


Figure 8.2 *Forces acting on cold structure due to rafter collapse when fire affects compartment 2.*

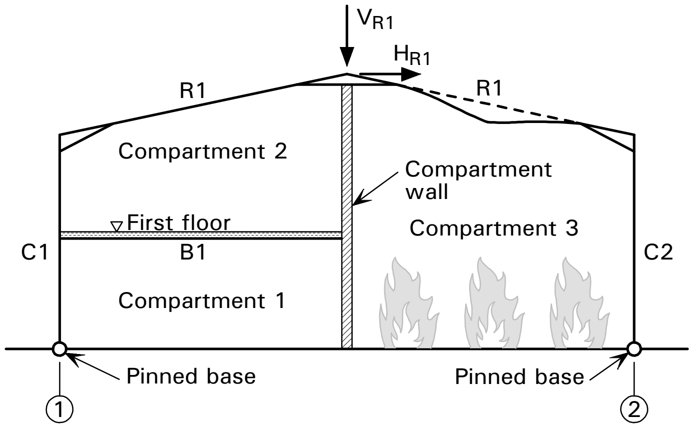


Figure 8.3 *Forces acting on cold structure due to rafter collapse when fire affects compartment 3.*

Summary of design requirements

Column C1: Should be fire-protected and should have a moment resisting base.

Column C2: No fire protection required.

It is recommended that a prop be provided in the line of the compartment wall at each rafter position. This prop can also be designed to support the first floor beams and provide lateral support to the compartment wall. The connection between the prop and the rafter should not pick up any load from the rafter in normal temperature conditions but should be designed to support the applied fire loading on half the rafter span in fire conditions. Provision of such a prop prevents large deflections of the rafter, such as those experienced in fire, affecting the integrity of the compartment wall. Any alternative solution without the inclusion of a prop needs to ensure the stability and integrity of the compartment wall and avoidance of disproportionate collapse by other means.

Case 1b: Boundary condition on gridline 2

If the boundary condition occurs on gridline 2, as shown in Figure 8.4, the following recommendations apply. The base of column C2 should be moment resisting and column C2 should be fire-protected to the same standard as the wall it supports.

Consideration should be given to the effect of rafter collapse on the stability of the section of the building not affected by fire. Particular attention should be given to the spread of fire between the two sections, the stability of the compartment wall and the possibility of disproportionate collapse.

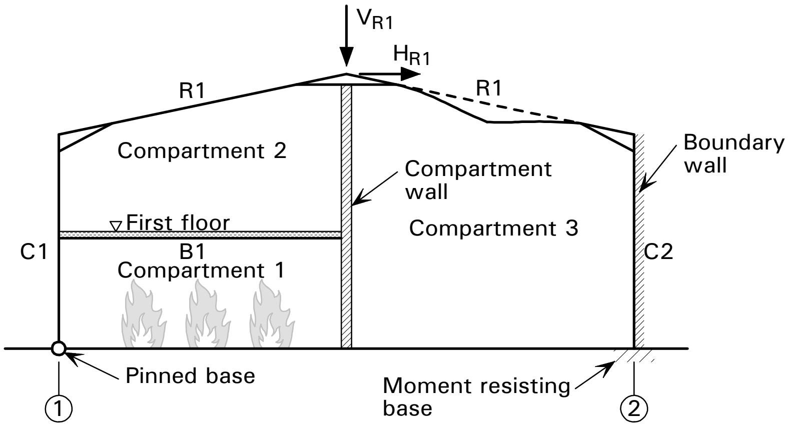


Figure 8.4 *Case 1b: Boundary condition on gridline 2*

To prevent rafter deflection in the fire condition affecting the integrity of the compartment wall, it is recommended that a prop be provided in the line of the compartment wall and designed to support the fire half design loading on the rafter. To maintain the stability of compartments 1 and 2, the transfer of the horizontal force back to the foundation should be considered. This can be achieved by portal action utilizing the end connections to beam B1, as described earlier, or by using a moment resisting base on column C1.

Summary of design requirements

Column C1: Not fire-protected may have a moment resisting base for stability.

Column C2: Fire-protected with a moment resisting base.

A fire-protected prop should be provided in the line of the compartment wall to pick up the vertical reaction from the fire-affected rafter. The connection between the rafter and the prop should be designed in such a way as to prevent the prop from becoming loaded in normal design conditions.

Case 1c: Boundary conditions exist on both gridlines 1 and 2

In this case the bases of columns C1 and C2 should be moment resisting and the columns should be fire-protected to the same standard as the walls which they support.

Provide a fire-protected prop to the rafter in the line of the compartment wall to pick up the vertical reaction from the fire-affected rafter and to prevent the deflection of the rafter from causing failure of the compartment wall. Given base fixity on both of the portal columns, no further consideration of lateral stability will be required.

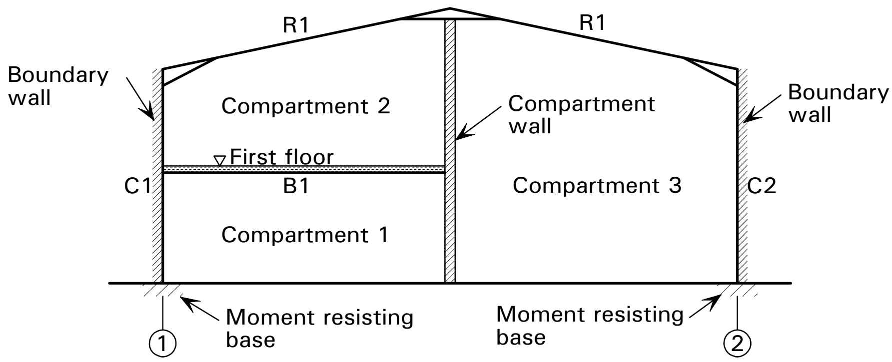


Figure 8.5 *Case 1c: Boundary conditions on Gridlines 1 and 2*

Summary of design requirements

Column C1: Fire protect this column and provide a moment resisting base.

Column C2: Fire protect this column and provide a moment resisting base.

Provide a fire-protected prop in the plane of the compartment wall to pick up the vertical reaction from the fire-affected rafter.

8.1.2 Case 2: Compartment wall parallel to frames

Office space is sometimes created by partitioning off an end bay of a building. The partition, which is treated as a compartment wall, runs parallel to the span of the main frame. The design considerations for fire are: maintenance of the stability and integrity of the compartment wall, stability of boundary walls, and fire resistance of steelwork supporting first floor.

The stability of the compartment wall will be affected by its position relative to the structural frame. In cases where the wall corresponds with the frame position, the stability and integrity of the wall will not be a problem as the frame being protected from the fire by the wall will not undergo significant deflection.

When the compartment wall occurs between gridlines, the deflection of the roof structure at the top of the wall may be more significant. Therefore, in this case more care must be taken to properly detail the top of the wall allowing for a reasonable deformation.

The steelwork supporting the first floor should have the standard of fire resistance required for the type of building being considered.

The stability of the boundary walls can be checked using the methods given for side or gable walls. However, the designer may also take into account the stability provided by the fire-protected steelwork at first floor level when considering the stability of the boundary walls. Depending on the layout of structural members, the requirement for moment resisting bases on the external columns could be avoided.

8.2 Calculation of horizontal reactions from the fire-affected rafters

When single storey buildings contain compartment walls running perpendicular to the span of the frame, a fire will only heat part of the rafter at a time. Therefore, it can be assumed that the horizontal reaction due to collapse of the fire-affected rafter will be lower than if the whole rafter were heated simultaneously.

The calculation method for mono pitched frames should be used to calculate the horizontal reactions for the rafter on either side of the compartment wall. This method will result in lower values of horizontal reaction, which reflects the reality of the situation more closely.

Conservatively, the horizontal reactions may still be determined by assuming that the whole of the rafter is heated simultaneously and employing the calculation method for symmetrical pitched rafters.

8.3 Fire protection of portal columns

The extent of the fire protection provided to the portal columns could be varied, depending on the reason for providing the protection in the first instance. If the column is protected because it is providing support to an external boundary wall, the fire protection should be provided over the full height of the column.

However, if the protection were provided to the column where it supports the first floor of a two storey section only, it would be reasonable to protect the column up to the level of the first floor only.

8.3.1 Mezzanine floors

Mezzanine floors will not have a direct effect on the stability of boundary walls. However, single storey buildings often contain mezzanine floors and their design in fire is a question that often arises.

The following advice is based on Approved Document B.

Steelwork supporting a mezzanine floor can be left unprotected, provided that the following conditions are met.

- The floor is used for storage only
- The floor is occupied by limited number of staff (no public access)
- Adequate means of escape is provided - at least one stairway within 4.5 m of a final exit from the building.
- The floor is not more than 10 m in width or length and does not exceed 50% of the floor area of the space in which it is situated.
- People using the storage area should be aware of any fire developing below

Otherwise, the steelwork will require fire protection in order to achieve the necessary fire resistance period. If an automatic detection and alarm system meeting the requirements of BS 5839-1[16] is installed, the floor size may be increased to not more than 20 m in width or length.

If the building is fitted throughout with an automatic sprinkler system meeting the requirements of BS 5306-2 for the relevant occupancy rating, together with the additional requirements for life safety, there are no limits on the size of the floor.

For large areas (50% or more of ground floor area or where the above conditions are not met), structural elements should have the necessary protection.

Where the main portal columns are also used to support the mezzanine floor, these will be considered as structural elements for the purposes of the regulations, regardless of whether or not a boundary condition exists. Figure 8.6 shows the extent of fire protection required for these columns. On the right hand, the column is shown with fire protection up to the underside of the haunch. If the column is supporting a boundary wall as well as the mezzanine floor, then this is the extent of fire protection that is required. However, if the column is not in a boundary condition and the fire protection is required only because the column supports the mezzanine floor, then the approach shown on the left of should be adopted. Both of these approaches are not strictly in accordance with standard practice, which normally requires the whole of an element of structure to be fire-protected, but both represent a reasonable approach that will not compromise the level of fire safety provided in the building.

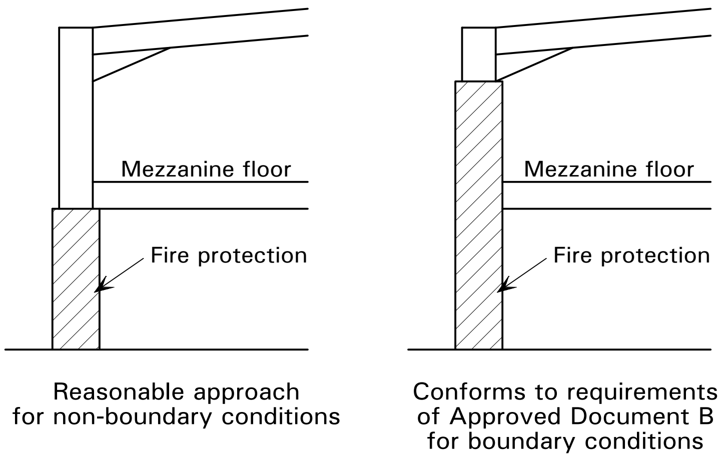


Figure 8.6 *Partial protection of columns*

9 REFERENCES

1 The Building Regulations 2000 (SI 2000 No 2531)
(Including Amendments 2001, 2002)
The Stationery Office

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Fire and steel construction: The behaviour of steel portal frames in
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12 BRITISH STANDARDS INSTITUTION
BS 5950: The structural use of steelwork in buildings
BS 5950-8:1990: Code of practice for fire resistant design

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15 BRITISH STANDARDS INSTITUTION
BS 6399 Loading for Buildings
BS 6399-3: 1988 Code of practice for imposed roof loads

16 BRITISH STANDARDS INSTITUTION
BS 5839 Fire detection and alarm systems for buildings
BS 5839-1:1988 Code of practice for system installation and servicing

APPENDIX A Mathematical model of rafter collapse

A.1 Derivation of overturning moment

The model assumes a worst-case scenario in which the rafter is inverted and acts like a catenary. However, a small allowance is made for the residual bending resistance of the rafter.

The geometry and the forces acting on the collapsing rafter are shown in Figure A.1.

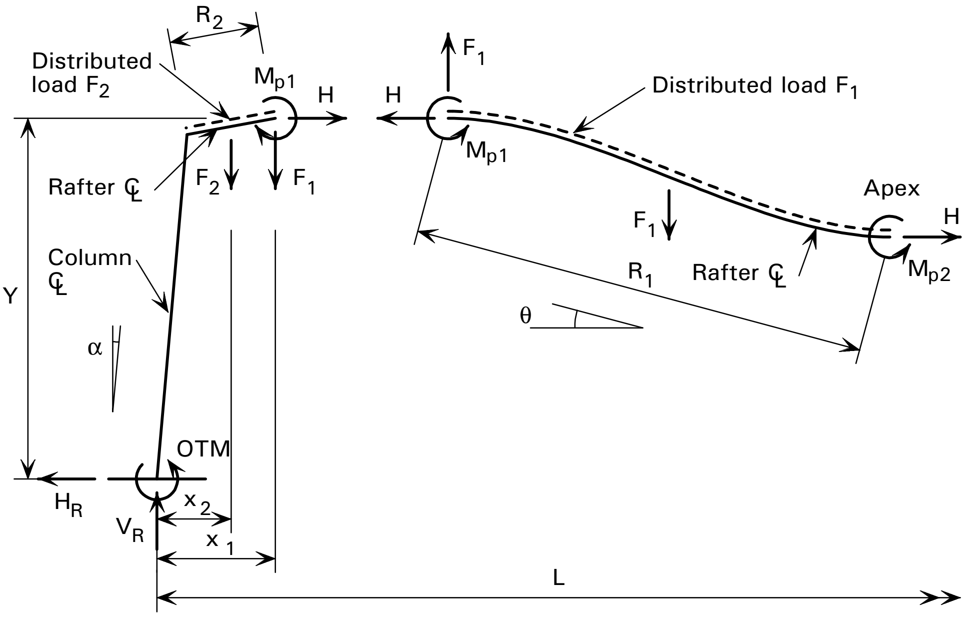

Figure A.1 *Forces and moments acting on the column and rafter.*

Considering vertical equilibrium, the vertical reaction on the column base is given as follows:

$$V_R = F_1 + F_2$$

Considering rafter equilibrium, taking moments about the apex:

$$M_{P1} + M_{P2} + H R_1 \sin\theta + \frac{F_1 R_1 \cos\theta}{2} = F_1 R_1 \cos\theta$$

where:

$$H = \frac{F_1 R_1 \cos\theta - 2(M_{P1} + M_{P2})}{2 R_1 \sin\theta}$$

and

$$H_R = H$$

For column equilibrium, the base overturning moment is given by:

$$OTM = HY + F_1 X_1 + F_2 X_2 + M_{p1}$$

where:

R_1 is the rafter length from end of haunch to apex including allowance for elongation

R_2 is the haunch length from centre line of column

Y is the height of end haunch
E cos α + R_2 sin $(q_0 - a)$

L is the span

E is the column height

q_0 is the initial rafter angle

X_1 is the horizontal distance from column base to end of haunch
E sin $\alpha + R_2$ cos $(q_o - a)$

X_2 is the horizontal distance from column base to end of haunch
E sin α + $^1/_2 R_2$cos $(q_o - a)$

a is the column deflection angle

q is the rafter sag angle
$\cos^{-1}\left(\frac{L - 2X_1}{2R_1}\right)$ for single bay frames
$\cos^{-1}\left(\frac{L - 2X_1 + E\sin a}{2R_1}\right)$ for multi bay frames

F_1 is the vertical load on the rafter length R_1

F_2 is the vertical load on rafter length R_2

V_R is the vertical reaction on column base

H_R is the horizontal reaction on column base

H is the resultant horizontal load in rafter

M_{P1} is the fire hinge moment at end of haunch

M_{P2} is the fire hinge moment at apex.

A.2 Assumptions made when using the model

To solve the above equations, some assumptions have to be made regarding frame geometry, loading and rafter moment resistance.

Rafter length, R1

Initially, the rafter length is the slope length along the rafter, from haunch to haunch. At elevated temperature the rafter will elongate. This is caused partly by thermal elongation and partly by additional mechanical strains that result from the reduction in elastic modulus of the steel due to elevated temperature. For the purposes of this model, a value of 2% has been assumed for this strain.

Column angle

It is assumed that the column deflection angle α is one degree. This determines the dimension X_1 and thus the inverted rafter profile (sag). A larger rotation could be assumed, provided that it can be demonstrated that the base can sustain that amount of rotation. It is reasonable to assume that a rotation of one degree can be achieved by elongation of the holding down bolts and some deformation of the base plate. If a larger rotation is assumed, the rafter sag will be greater and thus a slightly reduced horizontal reaction and overturning moment will be obtained.

Loading

See Section 2.4 for details of the reduced loading for the fire condition.

Fire hinge moments

The values M_{P1} and M_{P2} will be very much lower than the plastic moment resistance of the rafter at normal temperature. For frames utilising hot rolled universal sections as rafters, it is assumed that at the time of collapse they are both equal to 6.5% of the normal plastic moment of resistance of the rafter. This value represents the residual strength of steel at 890°C and experience has shown that its use gives reasonable results. For frames utilising tapered rafters M_{P2} is assumed to have this value but M_{P1} is further reduced by a factor of 0.85. This is due to the possibility of additional instability in deep rafters.

APPENDIX B Derivation of simple calculation method for symmetrically pitched portals

B.1 Simplifying assumptions

The full mathematical model for symmetrical pitched rafter described in Appendix A is tedious to use by hand. By making some further assumptions as to the geometry of the frame, the method can be simplified.

In the full model, the horizontal reaction on a symmetrically pitched frame is given by:

$$H = \frac{F_1\ R_1\ \cos q - 2\left(M_{P1} + M_{P2}\right)}{2\,R_1\ \sin q}$$

Rearranging this equation gives

$$H = \frac{F_1}{2\tan q} - \frac{\left(M_{P1} + M_{P2}\right)}{R_1\ \sin q}$$

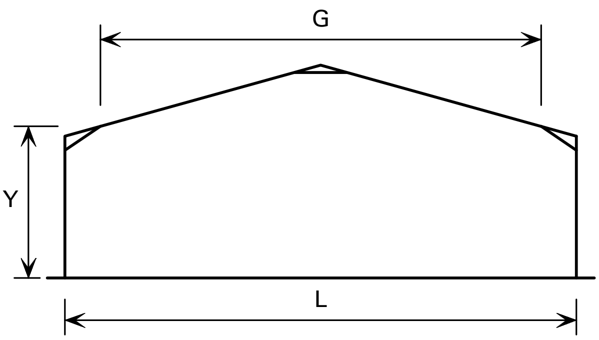

Figure B.1 *Frame dimensions*

The force on the main rafter F_1 may be expressed in terms of G and w_f

$$F_1 = \frac{w_f\ S\,G}{2}$$

where:

- w_f is the factored load on the rafter
- S is the frame spacing
- G is the horizontal dimension between the ends of the haunches

R_1 may be expressed in terms of G and the initial rafter pitch, q_0, assuming an elongation of 2%.

$$R_1 = \frac{G}{2\cos q_0} \times 1.02$$

The value of M_{P2} and M_{P1} are taken as 0.065 times the plastic moment of resistance of the rafter, M_P. The horizontal reaction can then be written as follows.

$$H = \frac{w_f \; S G}{4 \tan q} - \frac{0.255 \, M_P \, \cos q_0}{G \sin q}$$

Also

$$OTM = HY + F_1 X_1 + F_2 X_2 + M_{P1}$$

$$X_1 = E \sin a + R_2 \cos(q_0 - a)$$

$$X_2 = E \sin a + \frac{R_2}{2} \cos(q_0 - a)$$

If the column angle is taken to be one degree, the following good approximation for X_1 and X_2 may be made.

$$X_1 \approx \frac{Y}{60} + \frac{L - G}{2}$$

$$X_2 \approx \frac{Y}{60} + \frac{L - G}{4}$$

Then it can be shown that

$$F_1 X_1 + F_2 X_2 = w_f \; S G Y \left(\frac{L}{120 G} + \frac{L^2 - G^2}{8 G Y} \right)$$

$$HY = w_f \; S G Y \left(\frac{1}{4 \tan q} \right) - M_{pr} \left(\frac{0.255 Y}{G} \times \frac{\cos q_O}{\sin q} \right)$$

The expression for the overturning moment becomes:

$$OTM = w_f \; S G Y \left(\frac{1}{4 \tan q} + \frac{L}{120 G} + \frac{L^2 - G^2}{8 G Y} \right) - M_R \left(\frac{0.255 Y}{G} \times \frac{\cos q_O}{\sin q} - 0.065 \right)$$

This may be further simplified by making the following approximations. For a haunch length of about 10% of the span:

$$\frac{L}{120 G} = \frac{1}{96}$$

The rafter sag angle, q, is given by

$$q = \cos^{-1} \left(\frac{L - 2 X_1}{2 R_1} \right)$$

$$q = \cos^{-1} \left(\frac{(G - Y/30) \cos q_0}{1.02 G} \right)$$

A study of a large number of frames using the full model has shown that this approximates to

$$q = \cos^{-1}\left(0.97\cos q_0\right)$$

The overturning moment can then be written as

$$OTM = w_f\ S\,G\,Y\left(A + \frac{B}{Y}\right) - M_P\left(\frac{CY}{G} - 0.065\right)$$

In which:

$$A = \frac{1}{4\tan q} + \frac{1}{96}$$

$$B = \frac{L^2 - G^2}{8G}$$

$$C = 0.255\frac{\cos q_0}{\sin q}$$

Where L, G, q and q_0 are all as defined above.

Values of the parameters A and C are given in Table 2.2.

B.2 Portal Frames with low span to height ratios

The derivation of the simplified design method for symmetrical pitched rafters includes the simplifying assumption $q = \mathrm{Cos}^{-1}(0.97\ \mathrm{Cos}q_0)$

This simplifying assumption was based on an evaluation of a large number of frames, all of which had a span/height ratio of 1.6 or greater.

In order to extend the application of the simple method to frames with low span/height ratios, a further study was made for of frames with span/height ratios in the range 1 to 1.6. For this range, a more accurate simplifying assumption of $q = \mathrm{Cos}^{-1}(0.96\ \mathrm{Cos}q_0)$ was derived. The difference in accuracy (relative to the values given by the full model) between the new assumption and the old is shown in Table B.1, for a span/height ratio between 1:1 and 2:1.

It can be seen that the new assumption for rafter sag angle gives much better agreement with the full calculation method for ratios up to 2:1. (For greater ratios, the new assumptions become unconservative and the old assumption is still appropriate.)

Table 2.2 therefore gives two sets of values for parameters A and C, one for span/height ratios greater than 2:1 and one for ratios between 1:1 and 2:1.

Table B.1 *Percentage difference between values of OTM using the simplified method and using the full model*

a) Old assumption (using factor of 0.97)

Rafter pitch	Span/Height Ratio			
	1.0	1.2	1.6	2.0
3°	40%	31%	21%	15%
6°	37%	28%	19%	13%
9°	32%	25%	16%	11%

b) New assumption (using factor of 0.96)

Rafter pitch	Span/Height Ratio			
	1.0	1.2	1.6	2.0
3°	19%	15%	6%	1%
6°	17%	14%	6%	1%
9°	16%	13%	5%	1%

Note: The positive percentages indicate that the OTM values given by the simplified model are greater, i.e. they are more conservative.

APPENDIX C Unprotected areas in small buildings

Clause 14.20 of Approved Document B contains a simple method for determining the maximum amount of unprotected area permitted in buildings not more than 10 m high. The following recommendations are based on Clause 14.20 and may be used as a guide. For any building or compartment more than 10 m in height, the methods set out in the BRE Report *External fire spread: Building separation and boundary distances*[7], can be applied. In either case, reference should be made to Approved Document B.

For a building or compartment (other than an open-sided car park) that is not less than one metre from any point on the relevant boundary, the extent of the wall facing the boundary that may be left unprotected against fire should not exceed the limiting value given by Table C.1. It may be noted that residential buildings more than 12.5 m from the boundary and shops etc. more than 25 m from the boundary do not require any protection.

Table C.1 *Maximum permitted unprotected areas in buildings not exceeding 10 m high*

Residential, office, assembly and recreational buildings		Shops, commercial, industrial, storage and other non-residential buildings	
Minimum distance between side of building and relevant boundary (m)	**Maximum total percentage of unprotected area**	**Minimum distance between side of building and relevant boundary (m)**	**Maximum total percentage of unprotected area**
1	8%	1	4%
2.5	20%	2	8%
5	40%	5	20%
7.5	60%	10	40%
10	80%	15	60%
12.5	100%	20	80%
		25	100%

Notes:

Intermediate values may be obtained by interpolation.

For buildings which are fitted throughout with an automatic sprinkler system, the amount of unprotected area may be doubled or the separating distance may be halved, subject to minimum boundary distance of 1 m.

In calculating the maximum unprotected area, small areas of less than 1 m^2 may generally be disregarded.

APPENDIX D Worked examples

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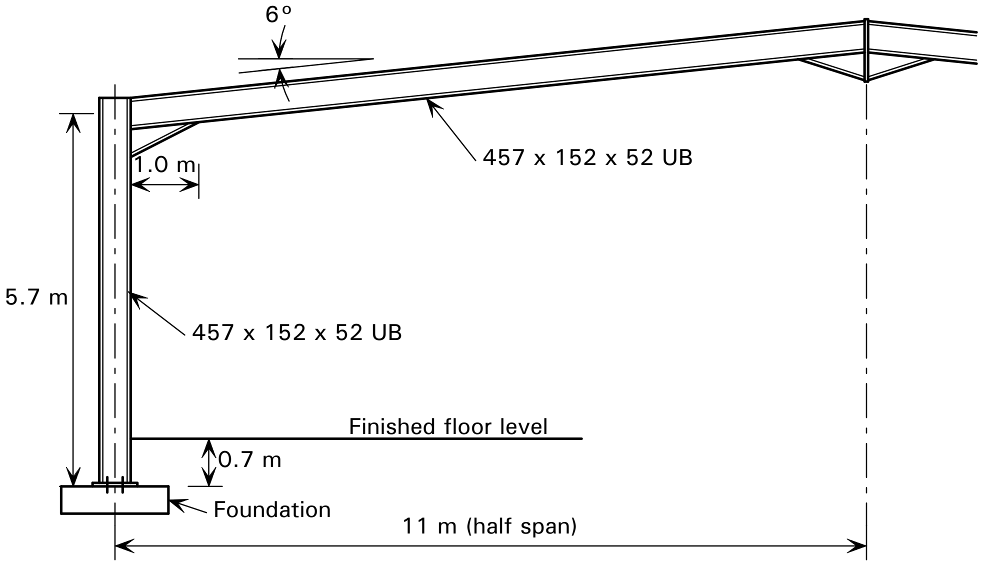
CALCULATION SHEET

Job No. BCF 858	Sheet 1 of 3	Rev A	
Job Title	Steel framed buildings in boundary conditions		
Subject	Worked Example 1: Single storey portal framed building with UB rafter		
Client SCI	Made by WIS	Date July 2002	
	Checked by GMN	Date July 2002	

2 BAY STEEL PORTAL FRAME WITH UNIVERSAL BEAM RAFTER

The building construction consists of a single span portal frame fabricated from UB sections, as shown in the following figure.



Span (L)	22 m
Height to eaves (E)	5.7 m
Frame Centres (S)	5 m
Rafter Pitch (q)	6°
Rafter Section	457 x 152 x 52 UB Grade S275
Column Section	457 x 152 x 52 UB Grade S275
Haunch length (H)	1.0 m
Height of end of haunch(Y)	5.76 m

Wall Construction

- Masonry wall 1 m high
- Steel sheeting above with unprotected sheeting rails and eaves beam
- W_D = 7 kN per frame

Roof cladding

Profiled steel sheet with foil liner and polystyrene foam insulation

Self weight 0.08 kN/m^2

Weight of roof structure

Purlins	0.03 kN/m^2
Rafter	0.10 kN/m^2
Total dead weight	0.21 kNm/m^2

Loading in fire at time of rafter collapse — 2.4.1

Steel sheeting	100%	0.07 kN/m^2	Table 2.1
Foam insulation	0%		
Foil liner	0%		
Purlins and rafter		0.13 kN/m^2	
Collapse load w_f		0.20 kN/m^2	

Plastic moment of resistance of rafter M_p = 301 kNm

Plastic moment of resistance of column M_c = 301 kNm

Distance between ends of haunches G = 20 m

Height of end of haunch Y = 5.81 m

Frame geometry parameter A = 0.93 — Table 2.2a

Frame geometry parameter C = 0.96 — Table 2.2a

Multiplication factor, K — 2.5.2

Span:height ratio $= \dfrac{L}{E} = \dfrac{22}{5.7} = 3.86 \quad \therefore$ use $K = 1.0$ — Table 2.3

$$\text{Parameter } B = \frac{L^2 - G^2}{8G} = \frac{22^2 - 20^2}{8 \times 20} = 0.525$$ — 2.5.1

Vertical reaction,

$V_R = ½ w_f SL + W_D = ½ \times 0.2\times\ 5 \times 22 + 7 = 18.0$ kN

Horizontal reaction

$$H_R = K\left[w_f SGA - \frac{CM_P}{G}\right] = 1\left[0.2\times5\times20\times0.93 - \frac{0.96\times301}{20}\right]$$

$$= 4.15 \text{ kN}$$

This must be checked against the lower limit based on 10% of plastic moment of resistance of the column.

$$H_R \geq \frac{M_c}{10Y} = \frac{301}{10\times5.81} = 5.18 \text{ kN}$$

\ H_R = 5.18 kN

Overturning moment

$$OTM = K\left[w_f SGY\left(A+\frac{B}{Y}\right)-M_p\left(\frac{CY}{G}-0.065\right)\right]$$

$$= 1\times\left[0.2\times5\times20\times5.81\left(0.93+\frac{0.525}{5.81}\right)-301\left(\frac{0.96\times5.81}{20}-0.065\right)\right]$$

$$= 54.2 \text{ kNm}$$

But $OTM \geq \frac{M_c}{10} = \frac{301}{10} = 30.1 \text{ kNm}$

$\therefore\ OTM = 54.2$ kNm

Longitudinal stability

The column, base plate, holding down bolts and foundations must be checked using the load factors given in Section 2.6. The longitudinal stability must be checked for compliance with Section 2.8. In this example, it is assumed that the requirements of Section 2.8.1 are met by correct detailing of the base. As the masonry wall is less than 75% of the height to eaves, the requirements of Section 2.8.2 may be met by designing the steelwork to BS 5950-1 or by designing the horizontal steel members such that:

$$\text{Combined strength in fire} \geq 0.025 \sum V_R \frac{\text{height of unprotected area}}{\text{height to eaves}}$$

(where the summation is over the number of frames)

V_R = 11 kN + weight of wall (W_D)

= 18 kN

Height of unprotected area = Height to eaves – height of dwarf wall

= 4.75 m

Assuming 10 frames

$$\text{Combined strength} \geq 0.025\left(18\times\frac{4.75}{5.75}\right)\times10 = 3.72 \text{ kN}$$

For a tensile stress of 0.065 times the yield stress of 275 N/mm^2

$$\text{Cross sectional area required} = \frac{3.72\times1000}{275\times0.065} = 208 \text{ mm}^2$$

The horizontal steel members providing longitudinal stability do not require fire protection.

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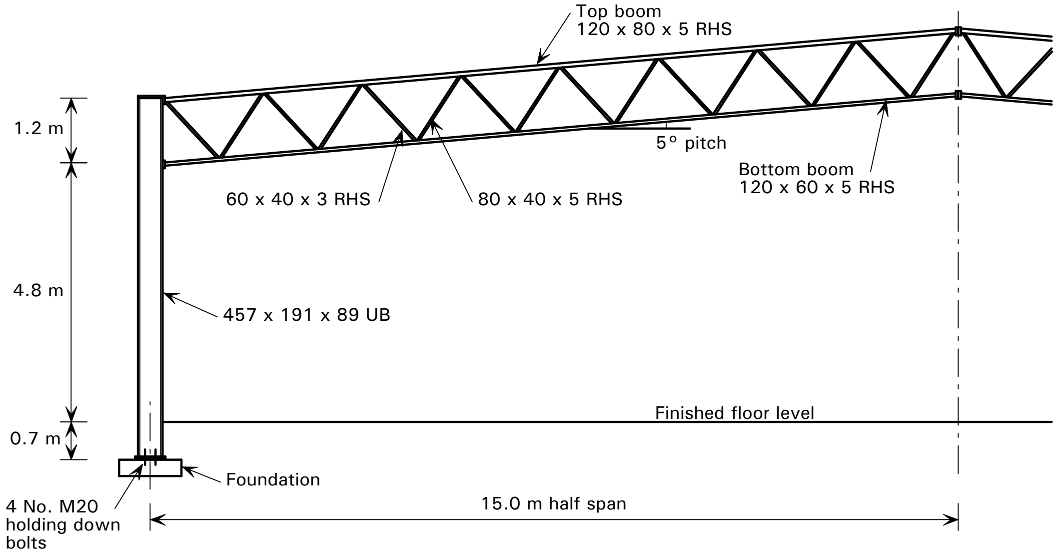
CALCULATION SHEET

Job No. BCF 858	Sheet 1 of 3	Rev A	
Job Title	Steel framed buildings in boundary conditions		
Subject	Worked Example 2: Single storey building with lattice rafter		
Client SCI	Made by WIS	Date July 2002	
	Checked by GMN	Date July 2002	

SINGLE SPAN STEEL FRAME WITH LATTICE RAFTER

The building construction consists of a single span lattice rafter as shown in the following figure.



Span (*L*) 30 m

Height to eaves (*E*) 6.7 m

Frame spacing (*S*) 6 m

Roof pitch (*q*) 5°

Haunch length (*H*) 0 m

Height to end of haunch, *Y* 6.7 m

The section sizes of the columns and the lattice rafter are shown in the figure.

Plastic moment of resistance of the column M_C = 534 kNm

Weight of roof structure

Purlins 0.03 kN/m²

Lattice rafter 0.10 kN/m²

Cladding 0.13 kN/m²

Total dead weight 0.26 kN/m²

Weight of wall cladding 0.16 kN/m²

Performance of roof cladding in fire — 2.4

The building is clad with rigid urethane cored panels consisting of an internal and external 0.5mm thick steel sheet.

Loading at the time of rafter collapse

Steel facings	100%	0.08 kN/m^2	Table 2.1
Urethane core	30%	0.02 kN/m^2	Table 2.1
Rafters and purlins		0.13 kN/m^2	
Collapse load w_f		0.23 kN/m^2	

Check the lattice rafters using the simplified method. — 2.5.5

Distance between haunches G = 30 m

Determine the frame geometry factors A, B and C

Span/Height = 4.5

Frame geometry parameter A = 0.95 — Table 2.2a

Frame geometry parameter C = 0.98 — Table 2.2a

Parameter B = 0 (the rafter has no haunches) — 2.5.5

Forces and moments on column bases — 2.5.1

Horizontal reaction

$$H_R = K\left(w_f\ SGA - \frac{C\,M_P}{G}\right) = 1.0\left(0.23\times6\times30\times0.95 - \frac{0.98\times0}{30}\right)$$

$$H_R = 39.33 \text{ kN}$$

Overturning moment

$$OTM = K\left[w_f\ S\,G\,Y\left(A + \frac{B}{Y}\right) - M_p\left(\frac{C\,Y}{G} - 0.065\right)\right] \geq \frac{M_C}{10}$$

As for a lattice rafter $M_p = 0$ and $B = 0$ the equation for OTM simplifies to: — 2.5.5

$$OTM = K\left[w_f\ S\,G\,YA\right] \geq \frac{M_C}{10}$$

$$OTM = 1.0\left[0.23\times6\times30\times6.7\times0.95\right]$$

$$OTM = 264 \text{ kNm}$$

Check that OTM is greater than 10% of the plastic capacity of the column

$$\frac{M_C}{10} = 53.4 \text{ kNm} \qquad \therefore\ OTM = 264 \text{ kNm}$$

Vertical Reaction

$$V_R = \frac{w_f SL}{2} + W_D = \frac{0.23 \times 6 \times 30}{2} + 0.16 \times 6 \times 6$$

$$V_R = 20.7 + 5.8 = 26.5 \text{ kN}$$

Design the column base plates and foundations. These should be designed to resist the forces and moment calculated above. 2.6 & 2.7

Check longitudinal stability 2.8

1. The column base plates are secured to the foundations with 4 equally spaced holding down bolts, 2.8.1
2. The horizontal members providing longitudinal stability for normal limit state conditions have been designed to meet the requirements of BS 5950-1. The frame can therefore be assumed to have adequate longitudinal stability for the fire limit state without needing to fire protected these longitudinal members.

Therefore, longitudinal stability is adequate.

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