



FIRE PERFORMANCE OF STEEL PORTAL FRAME BUILDINGS

M.W. Bong¹, A.H. Buchanan², R.P. Dhakal² & P.J. Moss²

ABSTRACT

This paper describes a study into the fire behaviour of steel portal frame buildings at elevated temperatures using the finite element programme SAFIR. The finite element analysis carried out in this report is three dimensional and covers different support conditions at the column bases, the presence of axial restraints provided by the end walls, several different locations and severities of fires within the building, different levels of out-of-plane restraint to the columns and the effect of concrete encasement to the columns. From a large number of analyses, it is shown that the bases of the steel portal frames at the foundations must be designed and constructed with some level of base fixity to ensure that the structure will deform in an acceptable way during fire, with no outwards collapse of the walls. The analyses also showed that it is not necessary for steel portal frame columns to be fire-protected unless the designer wishes to ensure that the columns and the wall panels remain standing, during and after the fire.

Introduction

Steel portal frame buildings with concrete tilt-up panels are a very common form of industrial building in New Zealand and Australia. They are formed by a series of parallel steel portal frames as the major framing elements which support the roof structure. Large clear spans of up to 40 metres or more can be achieved. Concrete tilt-up wall panels are commonly used as boundary walls as they allow fast erection and on-site fabrication. It is common to encase all or part of the steel portal frame column leg with concrete, or to use a reinforced concrete column for the lower part of the portal frame leg as shown in Figure 1.

In the past, concrete boundary wall panels were required to remain standing after a fire, but it is now considered acceptable for the panels to collapse inwards after a period of time, provided that they remain connected to each other. The inwards collapse of the walls can increase the fire separation distance to the relevant boundary and reduce the likelihood of horizontal fire spread by radiation. The inwards collapse may also extinguish the fire directly beneath the walls. However there remains concern that under fire conditions, the concrete panels may collapse outwards, creating a danger to fire-fighters and to adjacent property. This project investigates the fire behaviour of portal frame industrial buildings and explores design measures to achieve the goal of avoiding the outward collapse of the wall panels in fire.

In this study, the collapse deformation mode was considered to be either acceptable or unacceptable as shown in Figure 2.

¹ Arup Fire, Brisbane, Queensland, Australia

² Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand

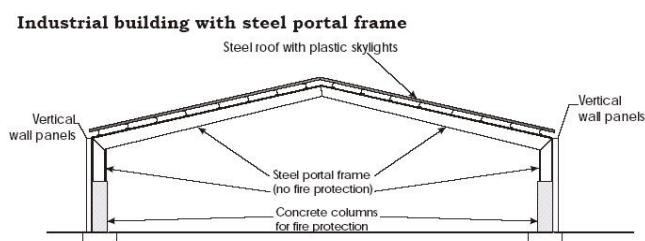
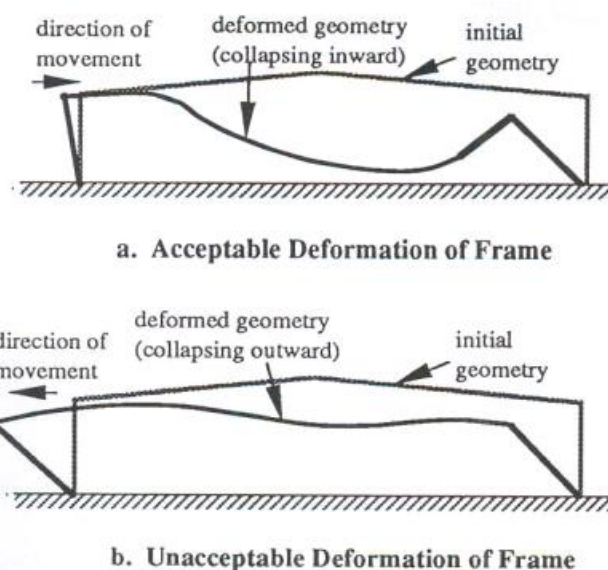


Figure 1 (above) Typical industrial buildings in New Zealand (Lim, 2000)

Figure 2 (right) Acceptable and unacceptable collapse deformation modes:
(a) inwards collapse, and (b) outwards sway collapse (O'Meagher et al, 1992)



Structural Details/Design

The structure studied is a typical industrial building formed by five parallel steel portal frames composed of 410UB54 sections as the major framing elements. The roof structure consisted of cold-formed DHS250/15 purlins and DB89/10 brace channels. The building was designed according to the New Zealand standards NZS 4203:1992 and NZS 3404:1997.

The building is 40 metres long by 30 metres wide and the roof was inclined at 8° . The steel frames had a span of 30 metres and were spaced at 7.2 metres. The columns were 6 metres high and the distance from ground level to the apex of the frame was 8 metres. The purlins were spaced equally at about 1.5 metres and spanned between the steel frames. The diaphragm action of the steel roof sheeting was ignored in the analysis but the self-weight was included. The loads applied to all the analytical models are the self-weight of the steel members and the steel roofing. The calculated load ratios for the steel portal frame with ideally pinned and fixed support conditions are 0.21 and 0.18, respectively (i.e. lower load ratio signifies better fire resistance).

The columns were assumed to be attached to the wall panels at the top and mid-height, and the end purlins were assumed to be supported on the end walls. The wall panels were not included in the analytical model but were represented by providing restraints to the columns and the ends of the purlins. These restraints prevented the out-of-plane displacement at the top and mid-height of the columns and are required under ambient conditions to reduce the effective lengths of the columns and to prevent buckling about the weak axis.

Frames with fixed and pinned bases were analysed as these provide the upper and lower bound of the base-fixity of the structure. However, the fully pinned bases of the frames are never achieved in reality and some degree of fixity will always be provided from the bolted connections at the supports. A portal frame structure with partial base fixity at the supports was also analysed [Bong 2005].

Two extreme cases were investigated in this project for the purlin support conditions to the end walls (Figure 3), and they are referred to here as either with or without purlin axial restraint. The most important difference between these two support conditions is the translational fixity in the longitudinal direction of the purlins (i.e. purlin axial restraint) at the locations of the end walls. The axial restraints in the steel purlins can be achieved provided the bolted end connections have sufficient axial load capacity. In a real building, the actual level of purlin axial restraint provided by the end concrete walls will certainly lie somewhere between the two extremes of zero and fully restrained which were modelled in this project.

In practice, it is common to provide fire protection to all or part of the steel portal frame column legs with concrete encasement. However, concrete encasement may fall off when exposed to very high temperatures or when the steel portal frame deforms excessively. In addition, when the concrete panels are trying to bow away from the supporting structures as they are exposed to high temperatures on one side (i.e. thermal

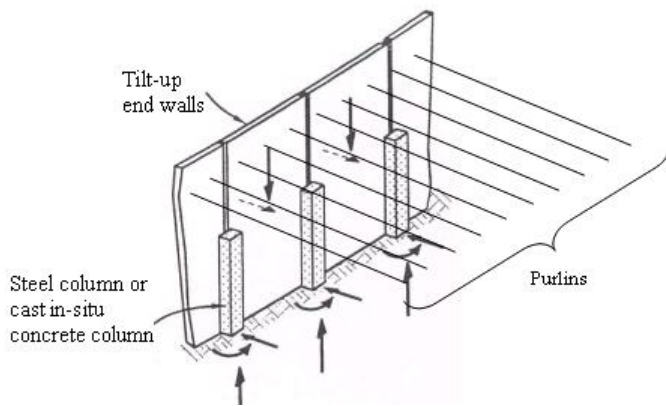


Figure 3 Purlins in the end bays attached to tilt-up end walls

bowing effects), the forces developed in the connections between the steel frames and the attached concrete panels will be larger due to the higher strength and stiffness of the protected steel columns (i.e. higher degree of restraint). If these connections fail, the wall panels could collapse outwards.

Analytical models with all the steel columns protected with cast in-situ concrete to either full or two-thirds of the height were analysed [Bong 2005, Bong et al 2007]. The analyses with columns fully encased in concrete served as the upper bound in terms of concrete protection to the column legs.

Analytical Modelling

Fire exposure

The fire curve used in most of the analyses in this study was the ISO 834 Standard Fire [ISO 1975]. However, the ISO fire is intended to represent fires in small compartments. The behaviour of a fire in a large compartment, such as warehouses or industrial buildings, is not the same as a small enclosure fire. These buildings usually have very high ceilings and large open spaces. The fire plume will have entrained a large amount of cold air when it impinges on the ceiling. The hot gases will continue to spread across the ceiling and similarly, cold air will be entrained into the ceiling jet. Therefore, the radiant heat flux from the upper hot layer may not be high enough to cause flashover. There is also likely to be venting through melted skylights and partial collapse of the roof in due course. For these reasons, the Eurocode External fire [EC1, 1994] (Figure 4) was used for some analyses.

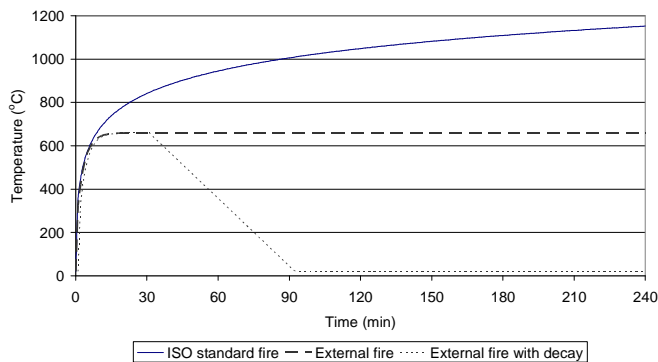


Figure 4 Time-temperature curves used in the analysis

Structural analysis

The main purpose of the study was to investigate the different failure modes anticipated for a typical portal frame industrial structure under fire conditions. Hence, this paper focuses on the fire behaviour of the complete building and a brief description of the 3D finite element model is given below.

Each of the steel portal frames was discretised into 40 beam elements [Bong 2005]. The nodes of the frames had seven degrees of freedom, i.e. 3 translations, 3 rotations and 1 warping.

Two nodes were created at the apex of the frame, one representing the left rafter and the other the right rafter. It was assumed that full compatibility could be achieved at the apex and warping was effectively transmitted between the two nodes. Similarly, two nodes were created at the knees to represent the column and the rafter. In this case, the nodes shared the same translations and rotations but the warping between the two nodes was not transmitted. At the column bases of the frame, the warping of the cross section was restrained by the endplate.

The ends of the purlins were joined to the nodes of the rafter (i.e. via master-slave relationships between these nodes) in a way that they behave similarly to fully fixed end supports but with rotation about the vertical axis freed. In practice, the purlins will be bolted to steel cleats which are welded to the top flange of the steel rafter. Some degree of fixity will be provided by the bolts to resist twisting about the longitudinal axis and in-plane deflection of the purlin. An assumption was made in the model that the bolts were able to provide full restraint against twisting about the longitudinal axis and in-plane rotation of the purlin. In terms of the warping of the purlins, it was neither transmitted to the rafter nor to the adjacent purlin since a small gap usually exists between the purlins at the support due to geometrical tolerances.

Results

Table 1 summarises the failure times in minutes and collapse modes of the analyses. The simulation end times in the table were obtained when SAFIR was unable to converge to a solution. The sway mode is that illustrated in Figure 2(b), whereas the inwards mode is illustrated by Figure 2(a). In the case of the upright mode, the columns remain straight and close to vertical, while the roof collapses inwards. For the catenary mode, the roof structure is supported by the purlins acting as suspension members between the end walls. It can be seen from Table 1 that the lesser temperatures reached in the External fire allow the portal frame structure to last longer than when exposed to the ISO fire. In the case of the External fire, the effect of axial restraint to the purlins is to greatly improve their performance, with no collapse at the end of 60 mins fire exposure. Figures 5-8 show the deflected shapes at the end of the analyses for the case of the External fire exposure. In Figures 5 & 7, the roof has deflected more than by the roof height since it was not possible to specify a limiting deflection at which to stop the analysis.

Table 1: Summary of analysis results

BASE FIXITY	Fire	ISO	ISO	External
	Column protection	None	Full height	None
	Purlin restraint			
PIN	No	14.1 Sway	15.9 Sway	18.0 Sway
PIN	Yes	19.6 Sway	17.2 Sway	60.0 No collapse
FIX	No	14.9 Inwards	14.7 Upright	26.9 Inwards
FIX	Yes	18.5 Catenary	19.6 Catenary	60.0 No collapse

Pinned Support Conditions

For a steel portal frame structure with pinned base connections, significant sidesway of the fire-affected frames will occur when the fire-affected roof structure (steel rafters, purlins and brace channels) begins to fail and the sway of the fire-affected frames will result in very large horizontal deflections at the top of the columns (i.e. possibly in excess of 1 m in the case of the ISO fire). After that, in the case where the purlins are not axially restrained, the roof structure will collapse to the ground and the analyses have shown that the collapsing rafters will subsequently pull the frames inwards (Figure 5), or the fire-affected roof structure will deform into a catenary with some sidesway action if the adjacent purlins are axially restrained (Figure 6). These failure modes are unacceptable and have been identified as the sway collapse mode because the large lateral deflections to one side could cause a side-sway collapse of one or more frames due to the P-delta effect related to the self weight of the walls.

Fixed Support Conditions

For a steel portal frame structure with bases fully fixed to the foundation, the deformation of the fire-affected roof structure (steel rafters, purlins and brace channels) is almost vertical without much sidesway. Immediately after the fire-affected roof structure starts to fail, the fire-affected frames will collapse inwards if the adjacent purlins are not axially restrained (Figure 7), or the fire-affected roof structure will deform into a catenary if the adjacent purlins are axially restrained by the surrounding structure (Figure 8). These failure modes are acceptable providing that the connections between the side walls and the supporting frames do not fail. Figure 8 shows that in the External fire, the fixed base portal frame structure undergoes little deformation when the purlins are axially restrained.

For the inwards collapse mode (i.e. no axial restraint to purlins), the initial outwards deformations of the steel columns are less than 200 mm at the top of the column and are solely due to the thermal expansion of the steel portal frame. When the fire-affected roof structure shows a snap-through failure mechanism and collapses to the ground, the columns will be pulled inwards along with the collapsing rafters. Therefore, the side walls will collapse inwards providing the connections between the walls and the supporting frame do not fail.

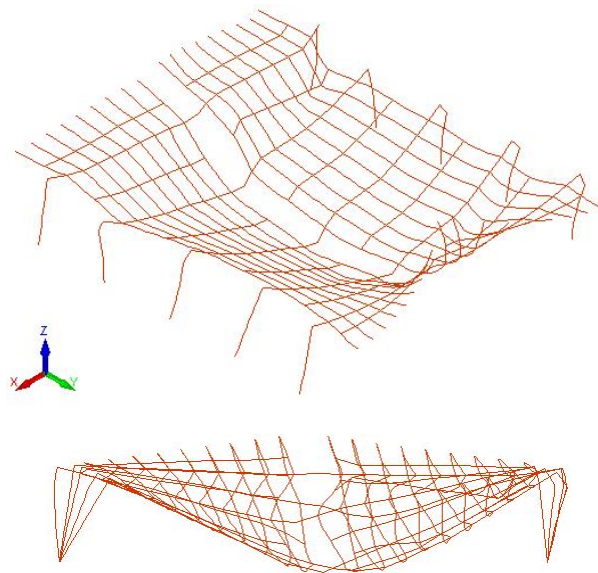


Figure 5 Sidesways collapse of the pinned support structure WITHOUT purlin axial restraint at time = 18.0 minutes (Scale =1x) in External fire

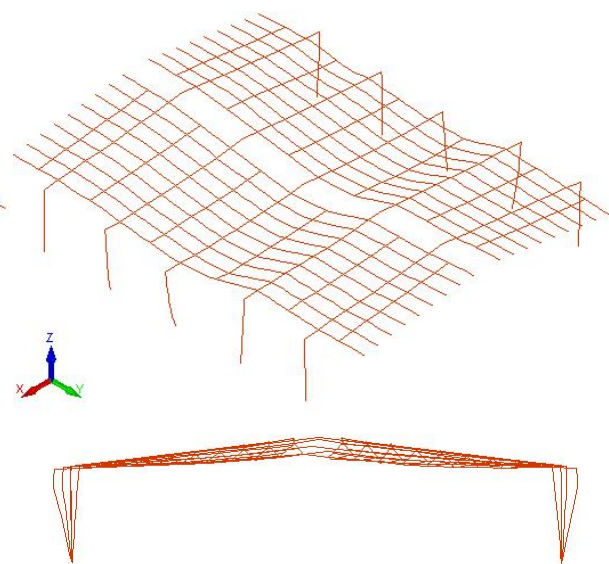


Figure 6 The pinned support structure WITH purlin axial restraint at time = 60 minutes (Scale =1x) in External fire showing some sideways movement but no collapse

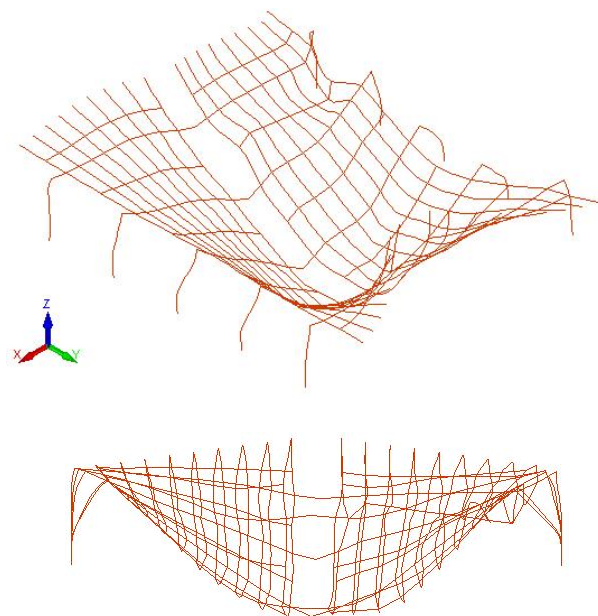


Figure 7 Inwards collapse of the fixed support structure WITHOUT purlin axial restraint at time = 26.9 minutes (Scale =1x) in External fire

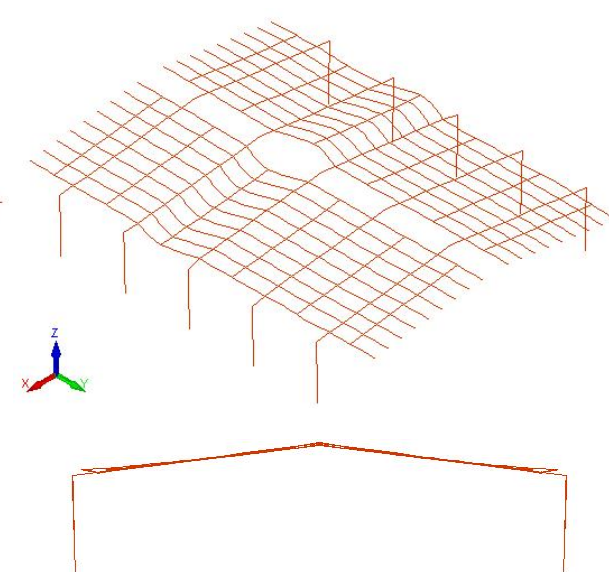


Figure 8 The fixed support structure WITH purlin axial restraint at time = 60 minutes (Scale =1x) in External fire showing catenary action with no collapse

Conclusions

The following conclusions can be drawn from the analyses carried out in this study:

- Most pin based frames fail in an undesirable sidesway mode.
- For the most common case of an ISO fire occupying the whole building, without strong axial restraint of the purlins and with out-of-plane column restraints provided by the side wall panels, structural collapse occurs at about 15 minutes.
- Full or partial base fixity, with column protection, gives good after-fire stability, with columns remaining close to vertical (hence much better reparability after a fire).
- Providing concrete encasement to columns gives no benefit if the column bases are fully pinned.

- In order to prevent collapse in an undesirable sway mode, the level of axial restraint of the steel purlins is less important than providing some degree of flexural fixity at the bases of the portal frame columns.

Design Recommendations

Support connections of the steel portal frames

The portal frame support connections must be detailed and designed to provide some level of rotational restraint, in order to prevent the sidesway of frames and outwards collapse of wall panels.

Passive fire protection to the column legs

Assuming that the recommendation of some base fixity will always be followed, providing fire protection such as concrete encasement to the columns can ensure that the columns will remain standing during and after the fire. If this design approach is adopted, stability of the external walls can be maintained in the post-fire condition, which may be desirable in many situations although not required by the codes.

Connections between the wall panels and the supporting frames

The wall panels must always be well connected to the supporting frames so that the outwards collapse of the panels, due to both thermal bowing of the concrete walls and outwards movement of the columns, can be prevented. This is regardless of whether or not the steel columns are fire protected. The new New Zealand Concrete Structures Standard NZS 3101:2006 requires at least two upper strong and well designed connections to the panels to ensure that the wall panels are well attached to the supporting columns. The connections near the top of the columns would have to withstand very high pullout forces. Apart from the top connections, additional connections should be located near the mid-height of the columns.

If multiple panels are used between the supporting frames, the panels must be well connected to each other such that they act as a complete unit. An eaves tie member is recommended to keep all the walls panels connected during a fire and the connections to the walls and supporting columns should be carefully detailed and designed to prevent outwards collapse of the individual panels.

References

- Bong, M.W. (2005), Structural Fire Performance of Steel Portal Frame Buildings, Master of Engineering (Fire) Thesis, University of Canterbury, Christchurch, New Zealand.
- Bong, M.W., Buchanan, A.H., Dhakal, R.P. and Moss, P.J. (2007), Fire Performance of Steel Portal Frame Buildings, Proc. 19th Australasian Conf. on the Mechanics of Structures and Materials, Balkema, Leiden, Netherlands.
- EC1 (1994), Eurocode 1: Basis of Design and Design Actions on Structures. ENV1992: Part 2.2: Actions on Structures Exposed to Fire, European Committee for Standardization, Brussels, Belgium.
- ISO (1975). "Fire resistance tests - elements of building construction", ISO 834-1975, International Organization for Standardization.
- Lim, L.C.S. (2000), Stability of Precast Concrete Tilt Panels in Fire, Fire Engineering Research Report 00/8, University of Canterbury, Christchurch, New Zealand.
- O'Meagher, A.J., Bennetts, I.D., Dayawansa, P.H., and Thomas, I.R and BHP Research Melbourne Laboratories (1992), Design of Single Storey Industrial Buildings for Fire Resistance, Journal of Australian Institute of Steel Construction, Vol. 26, No.2, Australia.
- Standards New Zealand (1992), General Structural Design and Design Loadings for Buildings, NZS 4203:1992, Wellington, New Zealand.
- Standards New Zealand (1997), Steel Structures Standard, NZS 3404:1997, Wellington, New Zealand
- Standards New Zealand (2006), Concrete Structures Standard, NZS 3101:2006, Wellington, New Zealand.