Structural performance of steel portal frame buildings subjected to fire

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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.

1 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 approximately 40 metres 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 also 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
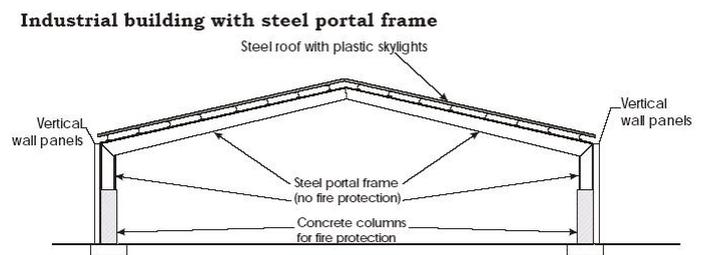

Figure 1 Typical industrial buildings in New Zealand (Lim, 2000)

avoiding the outward collapse of the wall panels in fire.

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

2 STRUCTURAL DETAILS/DESIGN

The structure studied in this project 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.
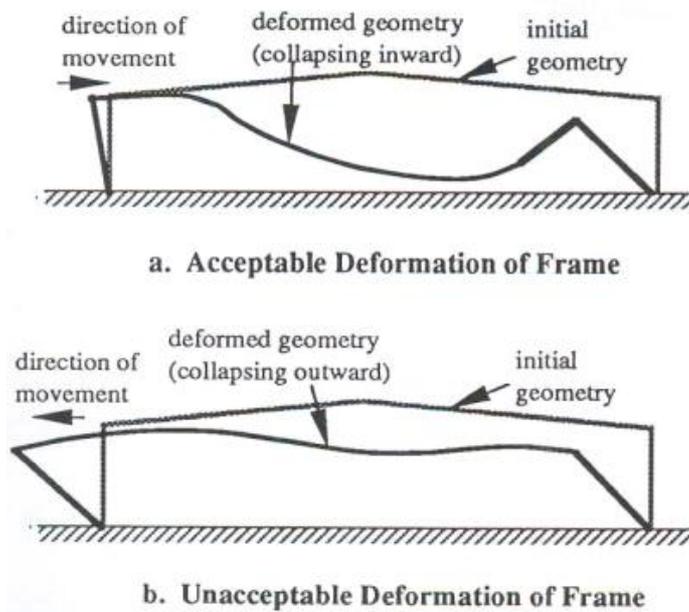
a. Acceptable Deformation of Frame

b. Unacceptable Deformation of Frame

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

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 pro-

vided 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.
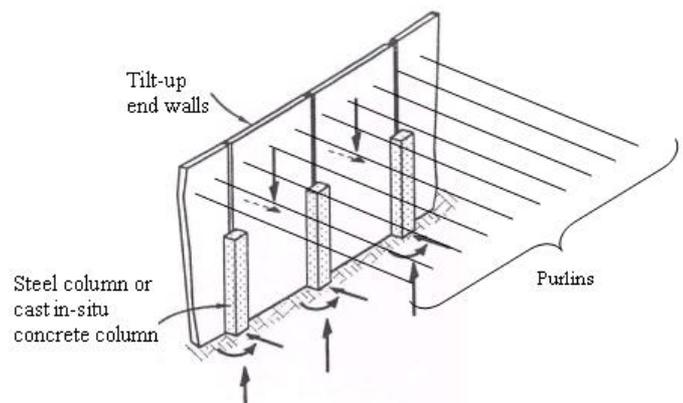

Figure 3 Purlins in the end bays attached to tilt-up end walls

In practice, it is common to fire protect all or part of the steel portal frame column legs with concrete encasement to provide additional fire resistance. 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 high temperatures on one side (i.e. thermal 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 walls 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]. Further, it is also common to fire protect the columns located close to neighbouring property while leaving the rest unprotected. The analyses with columns fully encased in concrete served as the upper bound in terms of concrete protection to the column legs.

3 ANALYTICAL MODELLING

3.1 *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. For this reason, the Eurocode External fire [EC1, 1994] (Figure 4) was used for some analyses [Bong 2005].
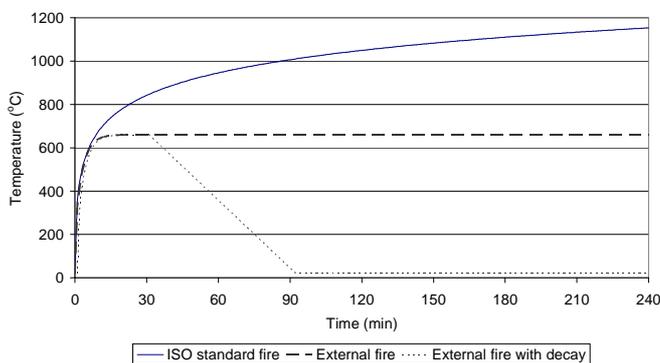

Figure 4 Time-temperature curves used in the analysis

3.2 *Structural analysis*

The portal frame structure was built-up progressively, with separate structural analysis performed on one single purlin, two and three dimensional single portal frames, part of a roof structure, and a two bay portal frame structure. This paper focuses on the fire behaviour of the completed building and a brief description of the 3D finite element model is given below.

Different support conditions at the column base, the presence of axial restraints provided by the end walls, different locations and severities of the fires, the effect of concrete encasement as passive fire protection to column legs and different levels of out-of-plane restraint to the columns were investigated [Bong 2005]. The main purpose of the analyses was to investigate the different failure modes anticipated for a typical portal frame structure under fire conditions.

Each of the steel portal frames was discretised into 40 beam elements. 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.

The torsional analysis calculates the elastic torsional stiffness and the warping function of the cross section at ambient temperature. In reality, the calculated value of elastic torsional stiffness will decease during the fire due to the increased temperature and the subsequent decrease in material stiffness. It is not possible to calculate the change in the torsional stiffness as a function of time in SAIFR and only a constant value can be entered. The calculated torsional stiffness was divided by a factor of two to represent the decreased stiffness at elevated temperatures [Franssen, J.M., 2004].

3.3 *Cleats at internal supports*

The brace channel is bolted on to proprietary steel plates, which are in turn bolted onto the web of the purlin. Similarly, it was assumed that the nodes between the brace channel and the purlin shared the same translations in the model (i.e. translation compatibility) and that the connection was able to provide restraints against twisting about the longitudinal axis and in-plane rotation of the brace channel.

4 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.

Table 1: Summary of analysis results

BASE FIXITY	Fire	ISO	ISO	ISO
	Fire size	100%	100%	100%
	Column protection	None	2/3 height	Full height
	Purlin restraint			
PIN	No	14.1 Sway	15.0 Sway	15.9 Sway
PIN	Yes	19.6 Sway	16.7 Sway	17.2 Sway
FIX	No	14.9 Inwards	14.2 Upright	14.7 Upright
FIX	Yes	18.5 Catenary	17.1 Catenary	19.6 Catenary

4.1 *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 5), or the fire-affected roof structure will deform into a **catenary** if the adjacent purlins are axially restrained by the surrounding structure (Figure 6). These failure modes are acceptable providing that the connections between the side walls and the supporting frames do not fail.

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.

4.2 *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). After that, the fire-affected roof

structure will deform into a catenary if the adjacent purlins are axially restrained (Figure 8), or 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 7). 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.

5 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 common column out-of-plane 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 vertical (hence much better reparability).
- 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.
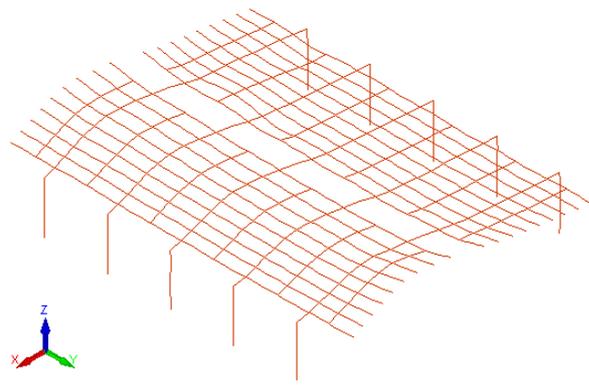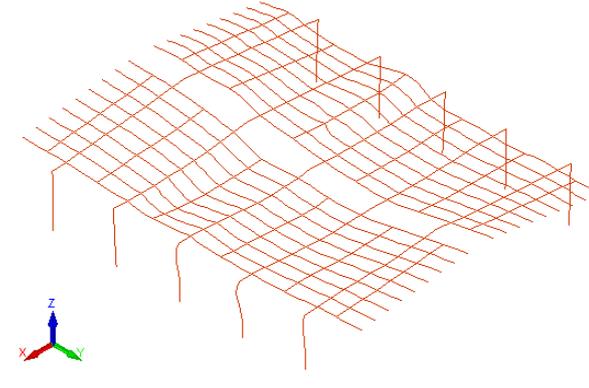
(a) Time = 14.90 minutes

(a) Time = 16.3 minutes

Horizontal deflection < 520 mm
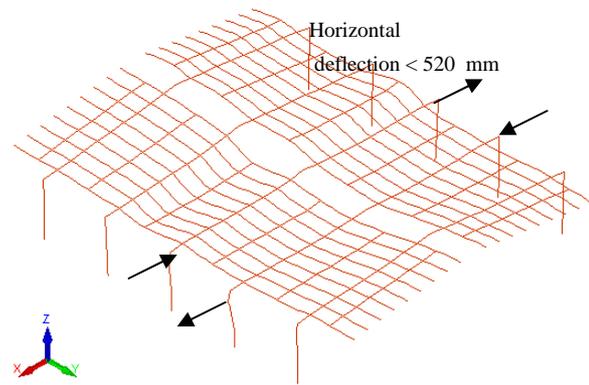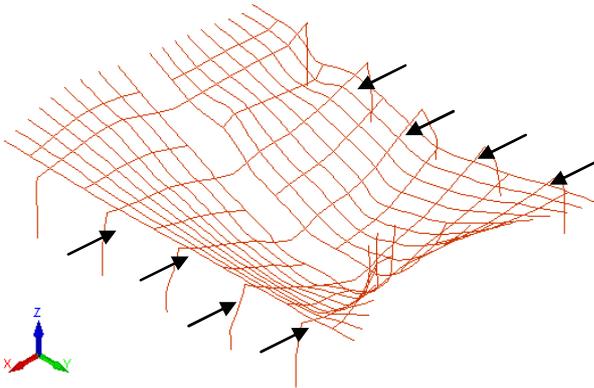
(b) Time = 14.92 minutes

(b) Time = 18.5 minutes

Figure 5 Inwards collapse of the fixed support structure WITHOUT purlin axial restraint imposed by the end walls and is fully involved in the ISO fire (Scale =1x)

Figure 6 Deflected shapes immediately before and after the rapid sagging of roof of the fixed support structure WITH purlin axial restraint imposed by the end walls and is fully involved in the ISO fire (Scale =1x)

6 DESIGN RECOMMENDATIONS

6.1 *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.

6.2 *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. This benefit does not apply if the portal frames have fully pinned supports.

6.3 *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.

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(a) Time = 14.12 minutes

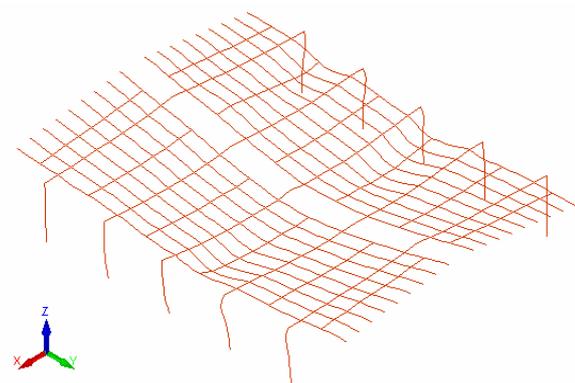Horizontal deflection > 1000 mm
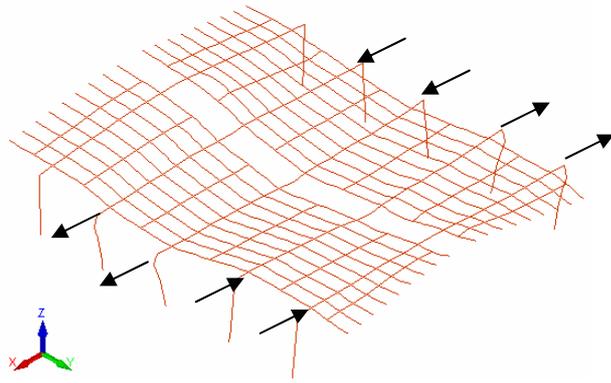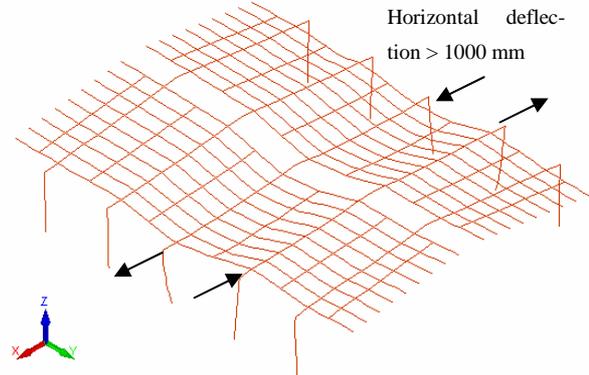
(a) Time = 13.9 minutes
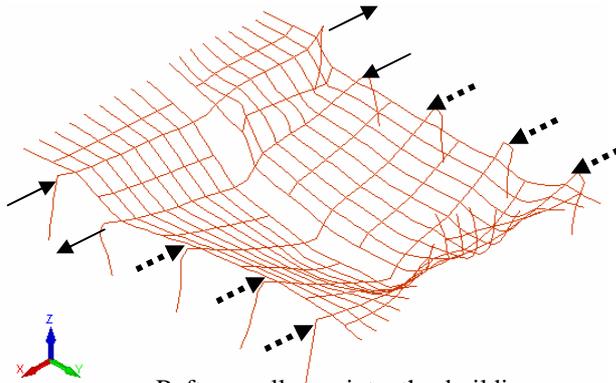
Rafters collapse into the building and to the ground pulling the steel columns inwards

(b) Time = 14.13 minutes

(b) Time = 19.6 minutes

Figure 7 Sideways collapse of the pinned support structure WITHOUT purlin axial restraint (Scale =1x)

Figure 8 Deflected shapes immediately before and after the rapid sagging of roof of the pinned support structure WITH purlin axial restraint (Scale =1x)