STRUCTURAL FIRE PERFORMANCE OF STEEL PORTAL FRAME BUILDINGS

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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 was three dimensional and covered 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 has been 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 wall panels are a very common form of industrial building in New Zealand and Australia. They consist of 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 economically using steel Universal Beams (UB). Concrete tilt-up wall panels are commonly used as boundary walls as they allow

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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 Fig. 1.

It has been observed in many real fire incidents that such buildings collapse or deform excessively when fires occur.
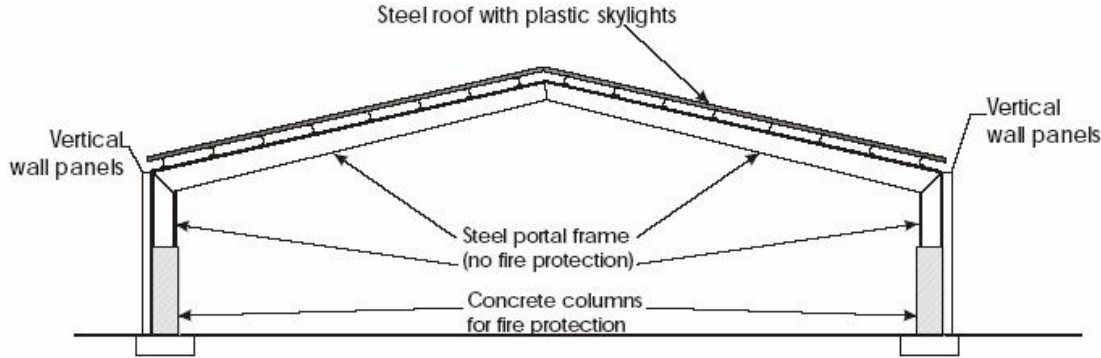

Fig. 1 - Typical industrial buildings in New Zealand[1]

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 provided that they remain connected to each other. However there still remains concern that under fire conditions, the concrete panels may collapse outwards, creating a danger to fire-fighters and to adjacent property.

Several acceptable or unacceptable deformation modes of such buildings under fire are shown in Fig. 2. For the frames collapsing into the building (**inwards collapse**), it can be seen that the resulting deformation will not endanger adjacent property or persons located outside
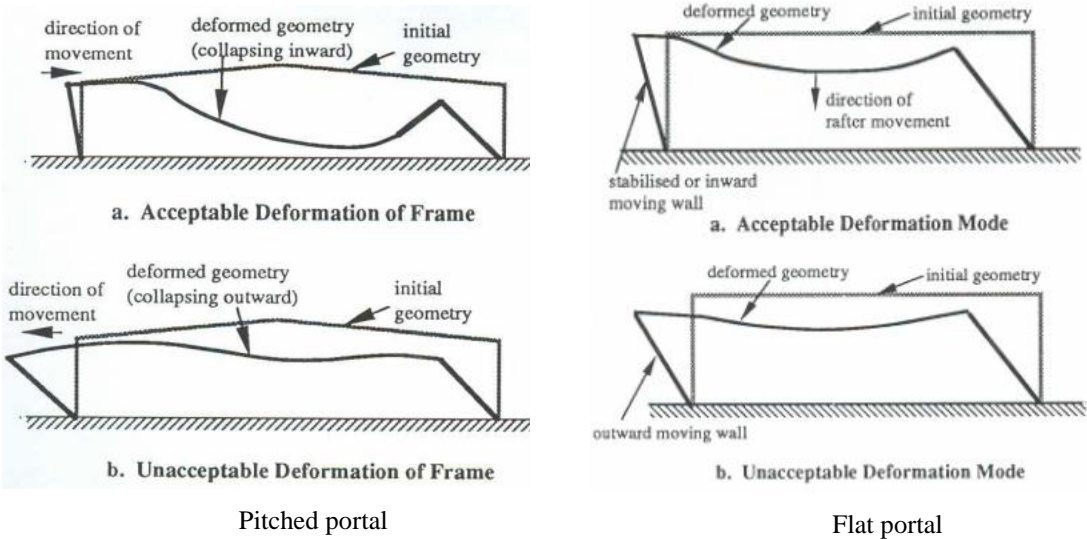

Pitched portal   Flat portal

Fig. 2 - Acceptable and unacceptable deformation modes[2]

the building provided that the boundary walls are tied together and fall inwards as a complete unit. 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. In contrast, frames that *sway sideways* may collapse outwards due to P-delta effects of the walls attached and may lead to adjacent property being damaged or persons outside the building being endangered.

This study describes an analytical investigation of the fire performance of steel portal frame buildings and establishes fire design guidelines for such buildings.

2. STRUCTURAL DETAILS/DESIGN

The structure studied in this project was an industrial building formed by five parallel steel portal frames composed of 410UB54 sections. 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[3, 4].

The structure was 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 steel sheeting was ignored in the analytical structure but the self-weight was included in the analysis. The columns of the frames 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.

Fix-fix and pin-pin supported frames were analysed as these provide the upper and lower bound of the base-fixity of the structure. However, fully pinned and fully-fixed bases are never achieved in reality as no real connection can provide full restraint or can allow the column base to rotate freely. Hence, a portal frame structure with partial base fixity at the supports was also analysed[5].

Under fire conditions, the collapse mechanisms of the walls depend on the performance of the supporting frames under elevated temperatures, providing that the connections between the walls and the frame do not fail.

The concrete panels themselves were not included in the models, but were represented by appropriate restraints to the steel members. 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.

The purlins in the end bays are connected to the concrete walls near the top of the walls (refer to Figure 3). The level of axial restraint provided by the end walls to the purlins is not well known and depends on many variables, such as the type of connection at the base, the amount of reinforcement and the thickness of the end concrete walls. In addition, it also depends on the supporting structures to which the walls are attached, the number of wall panels between the supporting structures and the types of connections used to join these structural elements together.

Two extreme cases of purlin axial restraint at these end wall connections were investigated, and they are referred to here as either *with* or *without* purlin axial restraint. 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 axial restraint will lie somewhere between the two extremes.
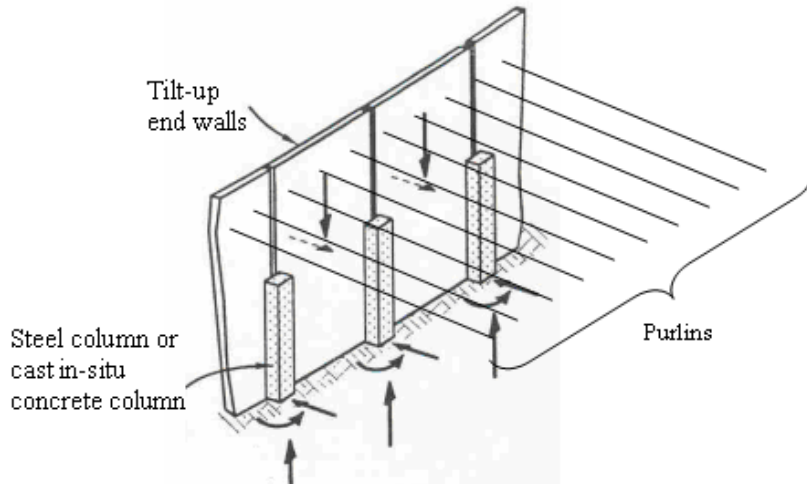
Fig. 3 Purlins in the end bays attached to tilt-up end walls

In practice, it is common to protect 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 during 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. 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 also analysed[5]. The analyses with columns fully encased in concrete served as the upper bound.

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).

3. SIMULATION MODELS

The first step is to perform thermal analysis on the structural members using appropriate fire curves. The fire curve used in most of the analyses in this study was the ISO 834 Standard Fire[6]. 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 was used for some analyses.

The Eurocode External fire curve assumes that the roof collapses and results in structural members exposed to lower temperatures (i.e. well ventilated fires with a maximum temperature of $660^{o}C$). The ambient temperature for both models was taken as $20^{o}C$.

A cooling phase was also introduced to the External fire. In Fig. 4, the External fire with a duration of 30 minutes is followed by a linear cooling phase. The linear decay rate used in the formation of this time temperature curve is from the Eurocode 1[7].
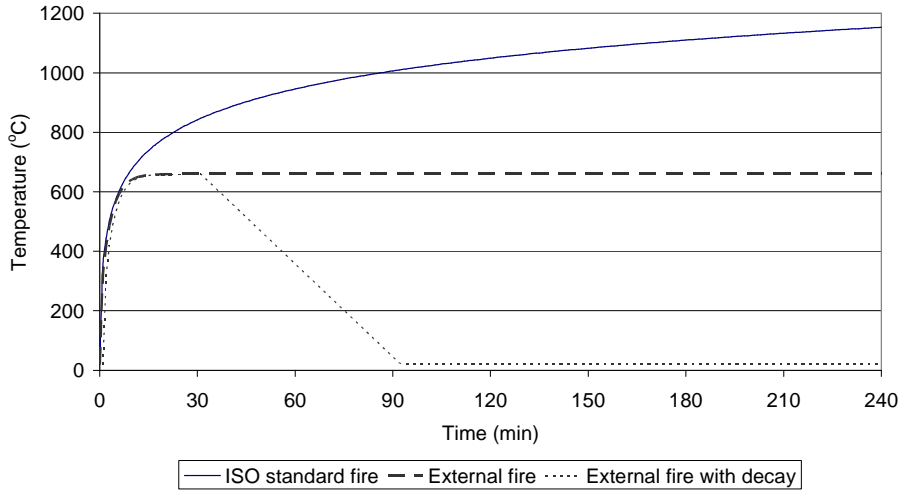

Fig. 4 Time-temperature curves used in the analysis

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 SAFIR 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[8].

The main purpose of the study was to investigate the different failure modes anticipated for a typical portal frame industal 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 as shown in Fig. 5. 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 (Fig. 6). 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.
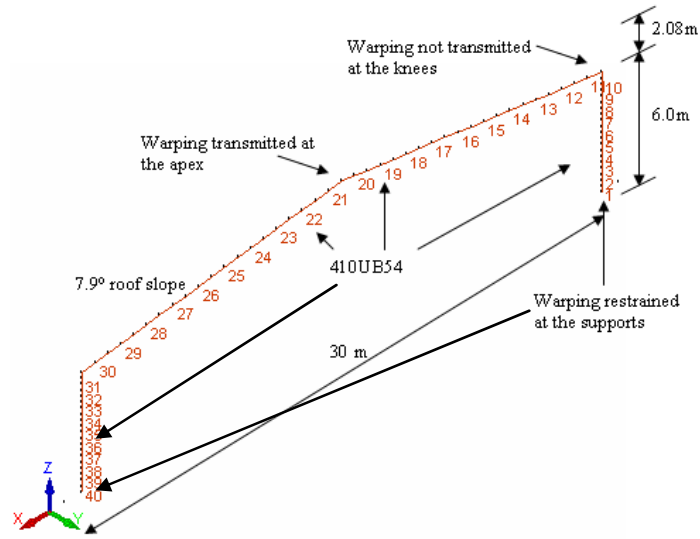
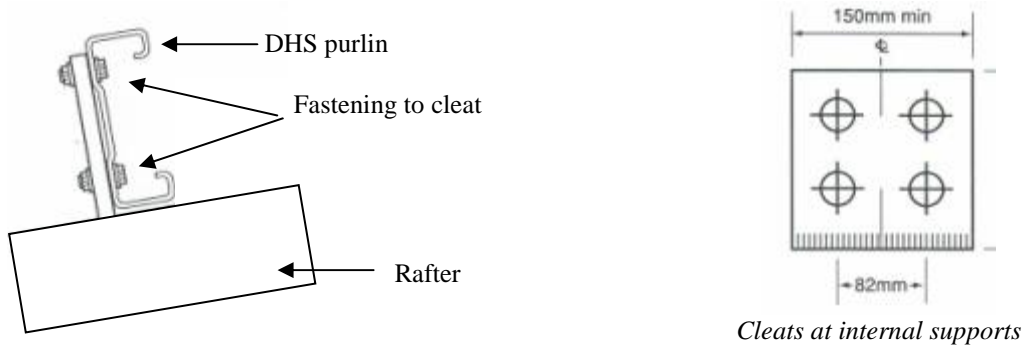Fig. 5 Finite element modelling of one of the steel portal frames

Cleats at internal supports

Fig. 6 Connection between purlin and steel rafter[9]

4. RESULTS

Table 1 summarises the analytically determined failure times and collapse modes of buildings with different base support conditions, and different purlin axial restraints. The simulation end times in the table were obtained either when SAFIR was unable to converge to a solution, or when the maximum time limit of 60 minutes (arbitrarily chosen) was reached. In all cases, the whole structure is subjected to the ISO fire. For most analyses, the bottom two thirds of the columns were concrete encased to provide fire protection.

4.1 Support Conditions at the column base

4.1.1 *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. 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 (Fig. 7), or the fire-affected roof structure will deform into a **catenary** if the adjacent purlins are axially restrained by the surrounding structure (Fig. 8). These failure modes are acceptable provided the connections between the side walls and the supporting frames do not fail.

6

Table 1: Summary of analysis results

BASE FIXITY	Fire	ISO	External	ISO	ISO	ISO
	Fire size	100%	100%	100%	100%	50%
	Column protection	None	None	2/3 height	Full height	None
	Axial restraint					
PINNED	No	14.1 Sway	18.4 Sway	15.0 Sway	15.9 Sway	15.1 Sway
	Yes	19.6 Sway	60 Sway	16.7 Sway	17.2 Sway	39.2 Catenary
PARTIAL FIXITY	No	15.6 Inwards		15.2* Inwards		
	Yes	15.9 Catenary		16.0* Catenary		
FIXED	No	14.9 Inwards	26.9 Inwards	14.2 Upright	14.7 Upright	31.2 Inwards
	Yes	18.5 Catenary	60 Catenary	17.1 Catenary	19.6 Catenary	39.7 Catenary

* Note: 2/3 concrete encasement applied only to the columns on one side*

(a) Time = 14.90 minutes

(a) Time = 16.3 minutes

Horizontal deflection < 520
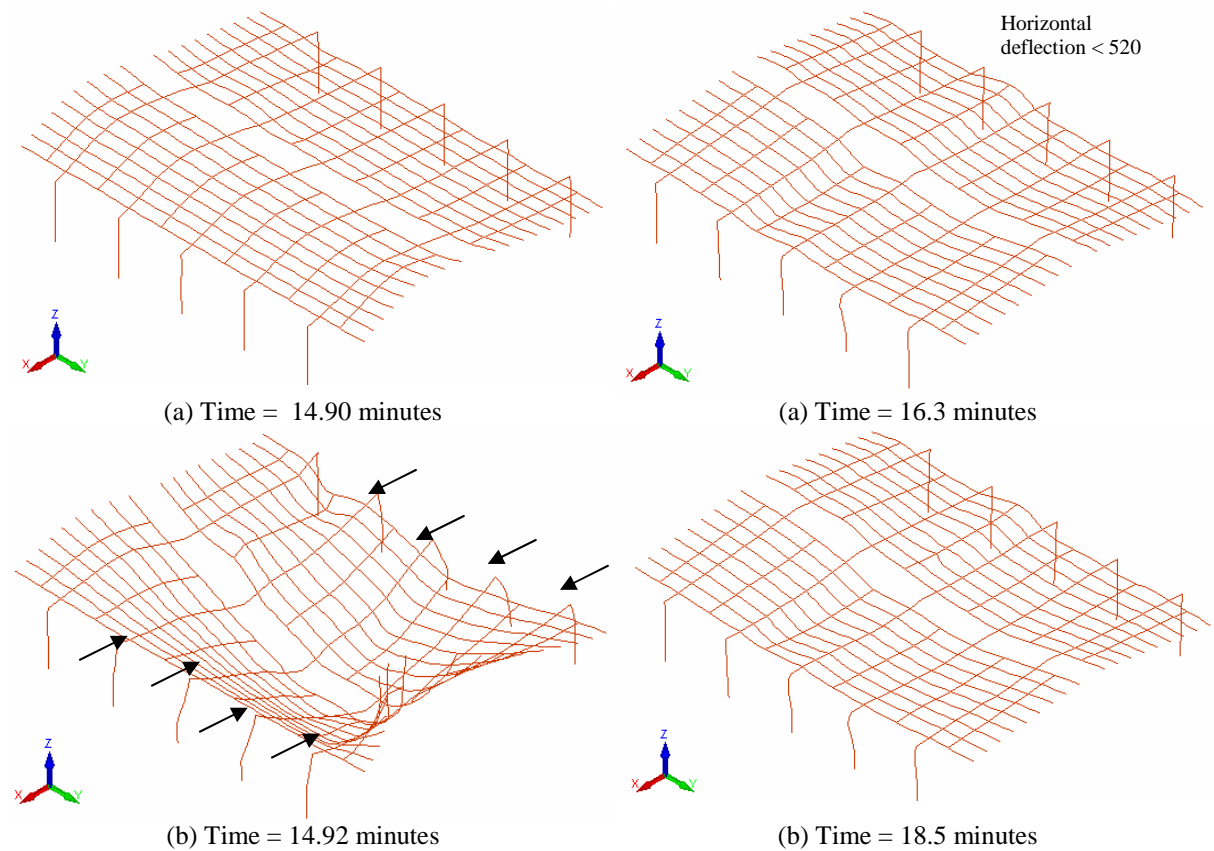
(b) Time = 14.92 minutes

(b) Time = 18.5 minutes

Fig. 7 Inwards collapse of the fixed support structure WITHOUT purlin axial restraint (Scale =1x)

Fig. 8 Deflected shapes immediately before and after the rapid sagging of roof of the fixed support structure and at collapse WITH purlin axial restraint (Scale =1x)

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 (Fig. 7), the columns will be pulled inwards along with the collapsing rafters. Therefore, the side walls will collapse inwards provided the connections between the walls and the supporting frame do not fail.

(a) Time = 14.12 minutes

Horizontal deflection > 1000

(a) Time = 13.9 minutes

Rafters collapse into the building and to the ground pulling the steel columns inwards

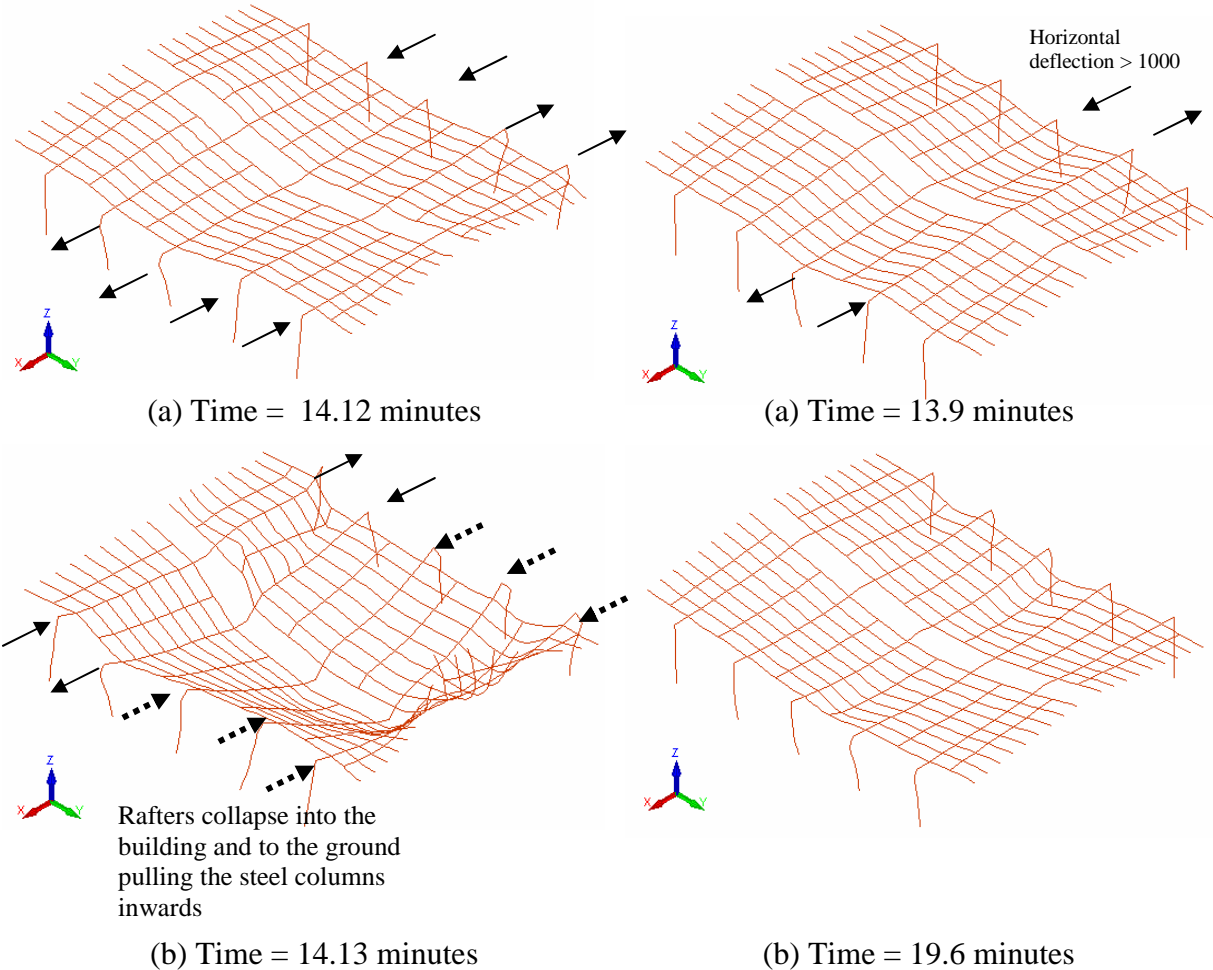(b) Time = 14.13 minutes

(b) Time = 19.6 minutes

Fig. 9 Sidesways collapse of the pinned support structure WITHOUT purlin axial restraint (Scale =1x)

Fig. 10 Deflected shapes immediately before and after the rapid sagging of roof of the pinned support structure WITH purlin axial restraint (Scale =1x)

For the **catenary** mode of failure (axially restrained purlins), the sagging of the fire-affected roof structure into a catenary shape will push the top portions of the columns outwards to some extent (i.e. up to 520 mm at the top of the column). Providing the connections to the walls panels do not fail, the walls can still be attached to the supporting frames and held in outwards inclined positions (Fig. 8). This is acceptable according to the New Zealand concrete code[10].

4.1.2   *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 (Fig. 10), 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 (Fig. 9). 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.

4.1.3   *Partially Fixed Support Conditions*

Most real buildings are designed and built with partially fixed portal frame bases. The collapse mechanisms of a structure with portal frames partially fixed at the base are similar to the structure with fully fixed support conditions. If the purlins fixed to the fire-affected steel frames are axially restrained by the surrounding structure, the structure will deform into a **catenary** as shown in Fig. 12. On the other hand, without axial restraint in the purlins, the portal frame columns and the attached wall panels will collapse **inwards** (Fig. 11) when the roof structure collapses to the ground.

4.1.4   *Passive Fire Protection*

For protected columns which are not pinned at the base and some fixity is provided by the support connections (i.e. either full or partial fixity), the concrete encased part of the columns will not deform excessively and will remain relatively straight during the fire. If purlin axial restraints are provided by the end walls, the structure will deform into a **catenary** (Fig. 12).

If the rafters collapse to the ground (i.e. no purlin axial restraint), the protected columns will not collapse inwards along with them and can still be standing **upright** after the fire (Figs 13 & 14). This is because the strength and stiffness of the concrete encased part of the steel columns are largely unaffected and the stability of the columns has not been affected. If the connections between the supporting frames and wall panels do not fail, the walls will be attached to the frames and remain standing during the fire. This is acceptable as long as the walls do not collapse outwards after the fire due to failure of the connections between the panels and the columns. However, the stability of the walls after the fire becomes an issue and the walls must resist wind loads as outwards collapse after the fire is also unacceptable.

The new New Zealand Concrete Structures Standard[11] will require that the connections be designed for a face load of 0.5 kPa applied to the concrete walls during the fire.

This study has shown that applying concrete encasement to columns which are fully pinned at the base will not improve the fire behaviour of the structure and **sidesway** of frames will occur resulting in outwards collapse of the columns and hence the walls.

2/3 of the columns protected with cast *in-situ* concrete
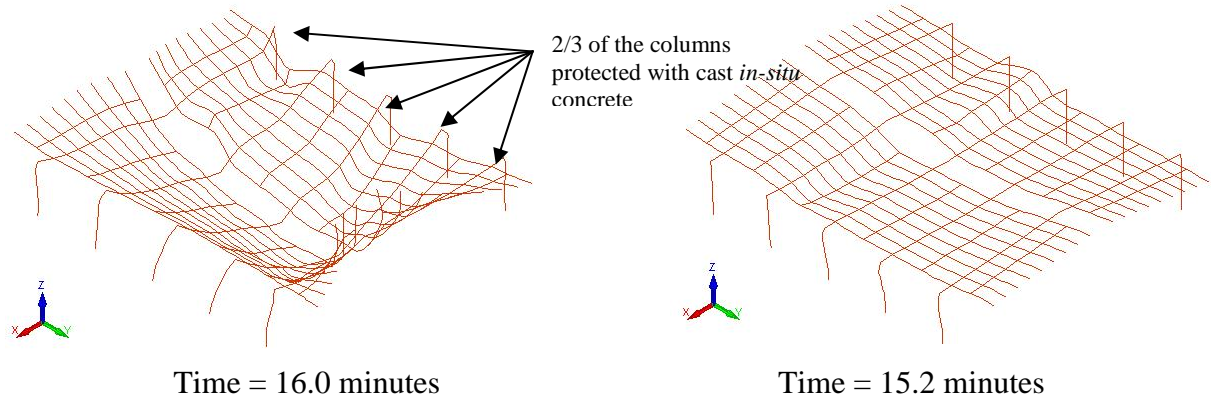
Time = 16.0 minutes

Time = 15.2 minutes

Fig. 11 Final deflected shape of the partially fixed support structure WITHOUT purlin axial restraint and 2/3 concrete encasement to right column legs (Scale = 1x)

Fig. 12 Final deflected shape of the partially fixed support structure WITH purlin axial restraint and 2/3 concrete encasement to right column legs (Scale = 1x)

2/3 of the columns protected with cast *in-situ* concrete

Time = 15.0 minutes
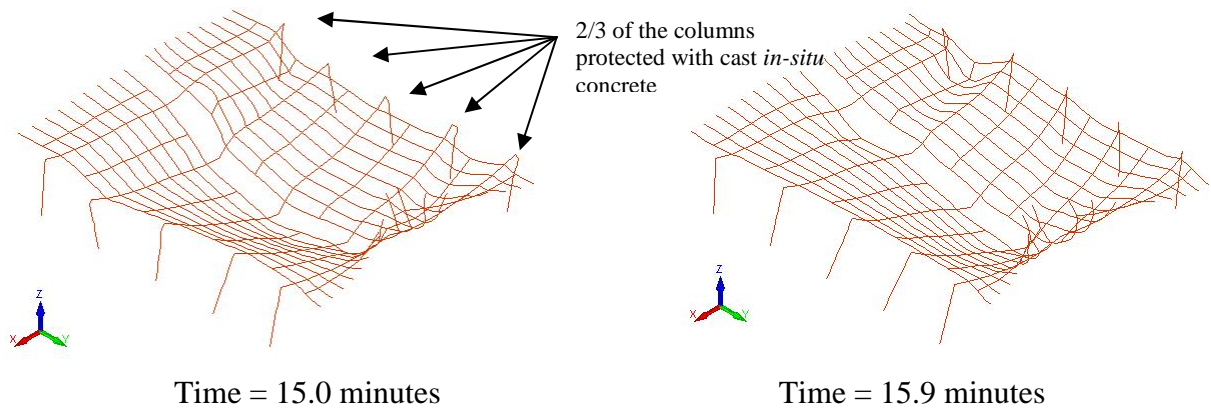
Time = 15.9 minutes

Fig. 13 Final deflected shape of the fully pinned support structure WITHOUT purlin axial restraint and with 2/3 concrete encasement to column legs (Scale = 1x)

Fig. 14 Final deflected shape of the fully pinned support structure WITHOUT purlin axial restraint and with full concrete encasement to column legs (Scale = 1x)

4.2 Steel Connections between the Side walls and the Supporting Frames

The top connections holding the walls to the supporting columns are very likely to fail due to high pull-out forces (Figure 15) as a result of the relative horizontal deflection between adjacent portal frames. This could possibly cause outwards collapse of the concrete walls if flexural capacity is not provided at the base, such as occurs with 'pinned' base walls. This suggests that there should be more connections between the top and bottom of the wall.

High pull-out forces in the top connection due to the relative horizontal deflection between frames
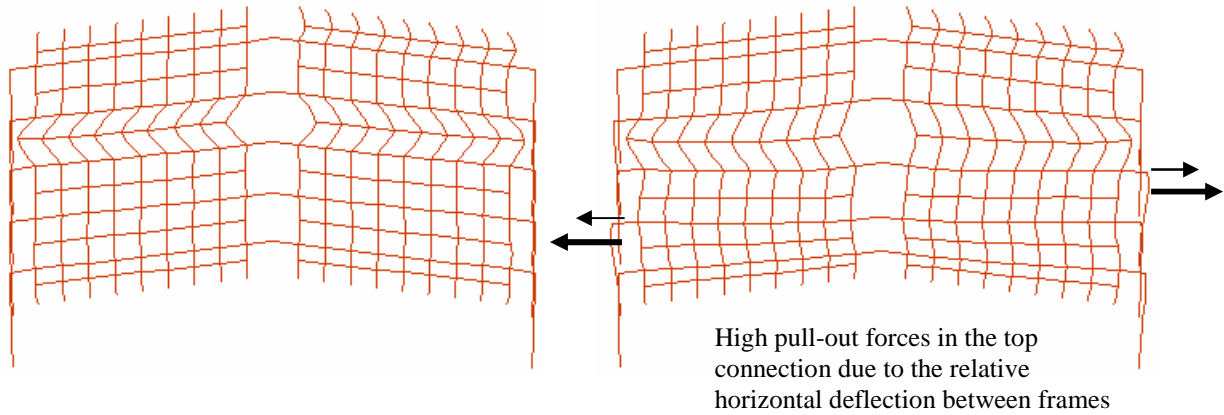
(a) Deflected shape immediately before rapid sag of roof

(b) Deflected shape immediately after rapid sag of roof

Fig. 15 Failure of top connection due to a pull-out mechanism - fully fixed frames with no concrete encasement (Scale = 1x))

5. CONCLUSIONS

The following conclusions can be drawn from the analyses carried out in this study:
- Most pin based frames fail in a sidesway mode and shpuld not be permitted.
- All partially fixed based frames have the same failure mode as fully fix based ones.
- 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 pinned.
- 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.

6. DESIGN RECOMMENDATIONS

Support connections of the steel portal frames
The portal frame base 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 and walls will remain standing during and after the fire.

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 3101:2005 will require 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.

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 individual panels.

7. REFERENCES

[1] Lim, L.C.S., Stability of Precast Concrete Tilt Panels in Fire, Fire Engineering Research Report 00/8, University of Canterbury, Christchurch, New Zealand, 2000.

[2] Bennetts, I.D. and O'Meagher, A.J., "Single Storey Steel-framed Buildings: Support of External Walls in Fire", BHP Structural Steel Development Group, Technical Note No.1, Australia, 1995.

[3] Standards New Zealand, General Structural Design and Design Loadings for Buildings, NZS 4203:1992, Wellington, New Zealand, 1992..

[4] Standards New Zealand, Steel Structures Standard, NZS 3404:1997, Wellington, New Zealand, 1997.

[5] Bong M.W., "Structural Fire Performance of Steel Portal Frame Buildings", Master of Engineering (Fire) Thesis, University of Canterbury, Christchurch, New Zealand, 2005.

[6] ISO, "Fire resistance tests - elements of building construction", ISO 834-1975, International Organization for Standardization, 1975.

[7] EC1, "Eurocode 1: Basis of Design and Designs Actions on Structures", ENV1992: Part 2.2: Actions on Structures Exposed to Fire, European Committee for Standardization, Brussels, Belgium, 1994.

[8] Franssen, J.M., *personal communication*, 2004.

[9] Dimond Industries, Hi-Span Design Manual, Manual No.9 in the Dimond Design Information Series, New Zealand, 1995.

[10] Standards New Zealand, Concrete Structures Standard, NZS 3101:1995, Wellington, New Zealand, 1995.

[11] Standards New Zealand, Concrete Structures Standard: Post Public Comment Draft 1a, Draft Number: DZ 3101.1 PPPC1a, Wellington, New Zealand, 2005.