

Fire Behaviour of Slender Precast Concrete Walls

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ABSTRACT

This paper describes the fire behaviour of industrial buildings which incorporate steel roof framing and precast tilt-up reinforced concrete wall panels. The walls were investigated as free-standing cantilevers, propped cantilevers, or attached to a simple frame of steel beams and columns. Analysis was conducted with the SAFIR program. The results show that tall and slender walls are likely to buckle or collapse outwards if they are not well connected to the steel frame, or if the building has inadequate resistance to transverse forces. Good performance can be obtained by providing fire-resisting connections between the panels and the steel frame, together with lateral resistance provided by a roof diaphragm or frame action.

KEYWORDS

Industrial buildings, precast concrete panels, fire, SAFIR, collapse

INTRODUCTION

The objective of this project is to investigate the fire behaviour of tilt-up precast concrete panels used as load bearing exterior walls in industrial buildings.

Tilt-up precast concrete construction is widely used in industrial buildings in New Zealand. The easy on-site fabrication of the panels and fast erection makes this the preferred method of construction for exterior walls. Precast concrete wall panels are cast directly on the concrete floor slab, and lifted into place by cranes after curing [1]. The tilt-up concrete panels are usually 100mm - 150mm thick, with a single central layer of reinforcing steel.

Code requirements

Most building codes require that the exterior walls remain standing during a fire, in order to provide a fire resisting barrier to protect adjacent property. The performance requirements of the New Zealand Building Code [2] require that occupants be able to escape from a building fire in safety and that fire-fighters be able to enter to conduct fire-fighting and rescue operations. The Building Code also requires that adjacent property be protected from fire spreading as a result of thermal radiation or structural collapse. The methods of achieving this performance are not specifically described in the Approved Documents [2].

The Building Code of Australia [3] has similar performance requirements, supported by specific details in the Approved Documents to ensure that exterior wall panels do not collapse outwards during a fire, which could cause danger to fire fighters and damage to neighbouring property.

Traditional construction

Traditional industrial buildings in New Zealand comprise steel portal frames with precast concrete panels on the exterior walls. The portal frames are typically spaced at 5 to 10 metres, supporting timber or steel purlins and thin steel roof sheeting with some translucent plastic skylights. During a fire in the building, the unprotected steel frames increase in temperature, leading to thermal expansion which tends to push out the exterior walls. Unreinforced brick walls or poorly attached concrete wall panels can fall outwards at this stage of the fire [4]. Increasing temperatures lead to downwards collapse of the steel roof framing, causing the exterior walls to be pulled inwards, possibly collapsing into the building.

Stability of the exterior wall panels is often achieved by fire-protecting the steel columns over their full length, or half length as shown in Fig 1, to provide lateral support to the precast concrete panels after the steel frames collapse during a fire as shown in Fig 2. Alternatively the wall panels are designed with a cantilever base so that the panels themselves remain standing independently when the portal frames collapse, assuming that the connection fails between the panel and the frame.

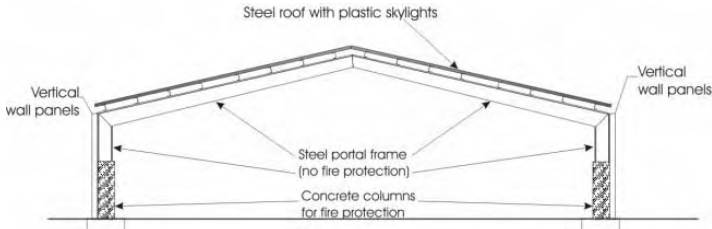


Fig 1. Traditional industrial building with half-height concrete columns.

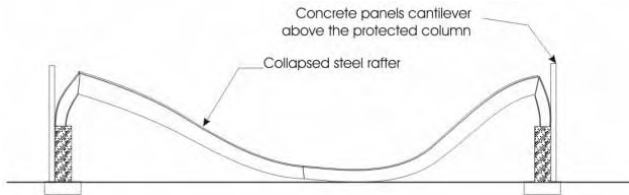


Fig 2. Fire behaviour of industrial building with half-height concrete columns.

Modern construction

There is a recent trend in New Zealand for modern industrial buildings to be built with long span beams and high ceilings, with tall and slender tilt-up precast concrete wall panels cantilevered from moment-resisting foundations at ground level, directly supporting the roof steelwork as shown in Fig 3. As well as resisting vertical loads from the roof, the concrete panels also provide in-plane resistance to lateral loads (earthquake and wind forces) with the roof bracing acting as a diaphragm. The clear spans of the rafters range between 15 to 30 metres and the rafters are spaced at 6 to 12 metre centres. These cantilever tilt-up panel buildings have been recently constructed with very slender panels up to 10 metres high. Panels with a high slenderness ratio have resulted in widespread concern about stability of the whole building during earthquake or fire exposure.

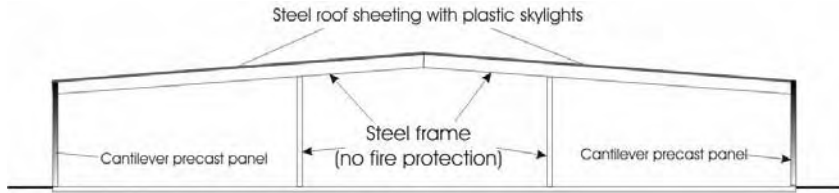


Fig 3. Construction of a typical industrial building with no exterior columns.

During a fire in the building, the heated steel rafters will expand and push the panels outwards, and the panels will tend to deform outwards themselves due to thermal bowing. After the rafters collapse, they will try to drag the wall panels back inward. There are several possible scenarios:

- If the steel frame has insufficient lateral resistance and there is not an effective roof diaphragm, the whole building could collapse sideways as shown in Fig 4.
- If there is a poor connection between the rafter and the panels, the connection will fail and the panels will cantilever from the foundations as independent fire separating walls. The cantilever concrete panels will be exposed to very high temperatures on one side, resulting in thermal bowing away from the fire, leading to instability and outwards collapse.
- If there is a good connection between the rafter and the panels, and resistance to sway is provided by diaphragm action of the roof, outwards collapse will not occur. Catenary forces from the rafter will cause a plastic hinge to form at the base of the panels, and the panels will collapse inwards as shown in Fig 5, which is much less dangerous than outwards collapse. It is increasingly recognised that inwards collapse of wall panels is acceptable, provided that the panels are tied together to prevent large gaps between them. Fig 6 shows the expected behaviour of panels exposed to a migrating fire in a building with an unprotected steel roof structure, and panels with no base fixity. The wall panels are often connected to each other with a steel eaves tie at the top, so that they will act in unison to prevent isolated inwards collapse of one panel.
- If the steel frame and the roof are fully fire-protected, they will provide lateral support to the top of the precast wall panels, so the panels act like propped cantilevers with a fixed base and pinned support at the top.

Most real cases will be a combination of these scenarios, with some interaction between the concrete wall panels and the steel frames.

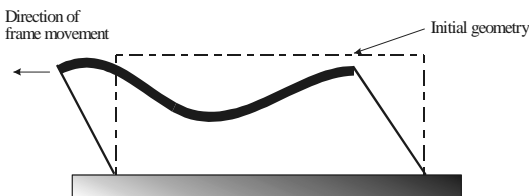


Fig 4. Sideways collapse of frame and walls.

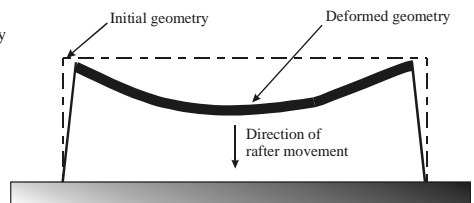


Fig 5. Inwards collapse of walls.

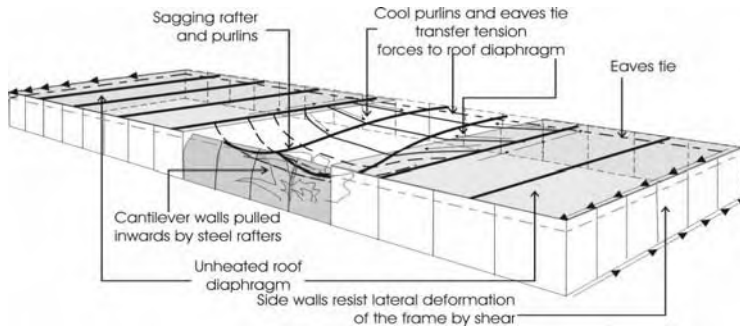


Fig 6. Behaviour of a building subjected to a migrating fire (Adapted from [12])

Most of these scenarios have been investigated in this study [5]. The research covers the behaviour of three similar structural systems:

- Free-standing cantilever walls (lower bound of expected behaviour)
- Propped cantilever walls (upper bound of expected behaviour). A fire-protected steel rafter and roof is assumed to restrain the lateral deflection at the top of the wall during a fire.
- Cantilever walls attached to an unprotected steel frame (intermediate behaviour between the lower and upper bounds).

CALCULATION METHODS

The calculation procedure for all fire exposed structures has three important components; the fire model, the thermal model, and the structural model.

Fire Model

The fire used in most of this study is the ISO-834 standard fire which is intended to represent fires in small compartments. The behaviour of fires in large open space industrial buildings is much harder to predict than in small compartments, because the fire may migrate through the building. Given the possibility of a migrating fire, the applicability of the ISO standard fire to the entire building is not realistic because different parts of the building would be exposed to different heat intensities at different times. For this reason, only parts of the steel frames were exposed to a fire at any time.

When severe fires occur in single story industrial buildings, the roofs usually collapse, in which case the exterior walls are exposed to a different type of fire, more like an open-air fire. In the absence of any better information, the Eurocode “external fire curve” has been used as a representation of such fires. The external fire is essentially a constant temperature of 660°C after a 10 minute growth period [6]. Recognising that fires will not continue to burn after the fuel is all consumed, more realistic fires were modelled by using the Eurocode external fire for a finite period of time, from 30 to 90 minutes, followed by a decay period.

Throughout this study it is assumed that the temperature exposure is uniform up the height of the walls. There are several reasons why the temperature might be greater at the top of a wall, but the effects of non-uniform temperature distribution were not investigated.

Heat Transfer Model

The thermal analysis in this project was conducted with a non-linear finite element program (SAFIR) developed by Franssen et al. [7]. Alternatively, the temperatures in a concrete section could be determined from a commercial finite element package or the simple hand method [8], but it is more appropriate to use an integrated thermal and structural model.

Structural Model

The deformations and stress conditions in a reinforced concrete wall exposed to fire cannot be determined without a sophisticated computer program which can accommodate both material and geometric non-linearities. The structural analysis in this project was conducted using SAFIR. Input of material properties and computational details are given by Lim [5]. Material properties are generally those in the Eurocode [9].

In previous studies, O'Meagher and Bennetts [10] developed a program to analyse the structural behaviour of concrete walls with pin supports, exposed to a fire on one side. This program was modified by Munukutla [11] to investigate different end supports to suit construction practices in New Zealand. Additional work has been published by O'Meagher, et al [12], O'Meagher [13] and O'Meagher and Bennetts [14], including design recommendations to prevent undesirable collapse of such structures onto neighbouring properties. Cooke [15, 16] and Cooke and Morgan [17] have investigated the fire behaviour of concrete cantilever walls, showing that thermal bowing deflections are significant for tall walls subjected to fires on one side.

RESULTS

Only a brief summary of the results is given here. Full details are given by Lim [5]. The output from the SAFIR analysis includes the temperature and stress distribution through the wall, deflections of the wall, and the bending moment and axial force at any point on the wall, all at any time step during fire exposure.

Free-standing cantilever walls

Free standing cantilever walls become hot on the fire side, so the concrete near the surface expands causing thermal bowing accompanied by compressive stresses near the hot face, balanced by tensile stresses in the central reinforcing and additional compressive stresses near the cool face of the wall.

Fig 7 shows the stress distribution through a typical wall at times up to 120 minutes. The tensile stresses in the reinforcing at the centre of the wall increase to reach the yield stress of the steel, well off the scale of the graph.

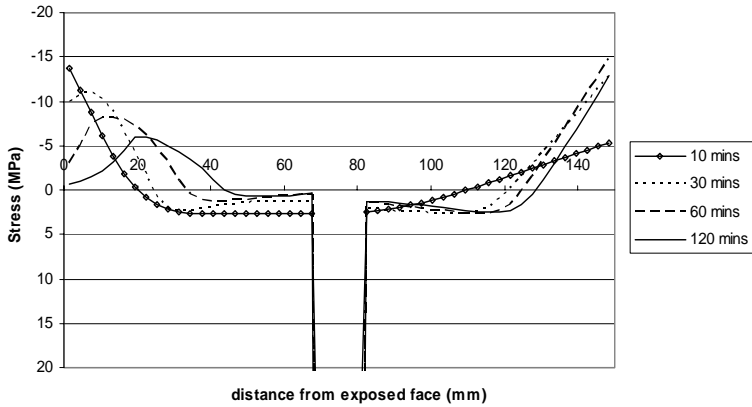


Fig 7. Stress distribution in typical cantilever wall.

Figure 8 shows the horizontal displacement at the top of 150 mm thick cantilever walls of various heights, exposed to the ISO-834 standard fire. Reinforcing is 0.67%, 430 MPa in one central layer. These calculated displacements are the result of thermal bowing, enhanced by P-delta effects. It can be seen that the 6 m high wall can resist the standard fire for two hours without collapse, when the top displacement is over one metre. As the wall height is increased, the time to failure decreases significantly, with the maximum sustainable displacement being 2.5 m for the 8 m high wall. Structural failure in all cases is accompanied by yielding of the central reinforcing steel at the base of the wall.

To investigate the effects of more realistic fires, the walls were exposed to the EC1 external fire for periods of up to 90 minutes, followed by decay at 625°C per hour. Fig 9 shows the horizontal displacement at the top of an 8 m high wall for several different fire exposures. It can be seen that the EC1 external fire produced smaller displacements than the ISO-834 fire, and the walls tend to return to their original shape after the fires go out. For taller walls the difference between the two fires is less significant, with collapse in many cases.

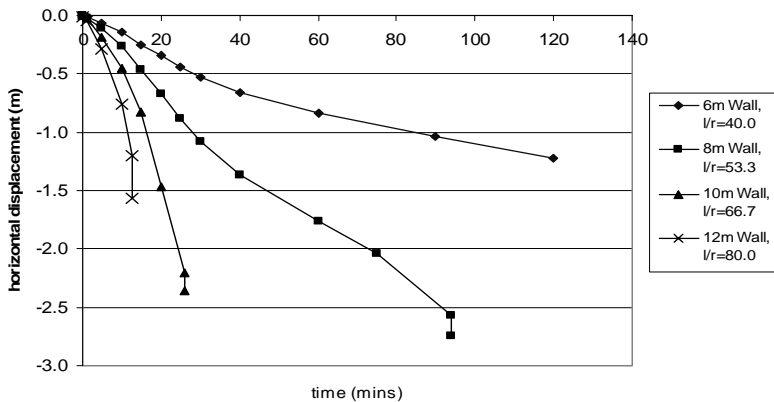


Fig 8. Comparison of horizontal displacements for cantilever walls of different heights, exposed to the ISO Standard fire.

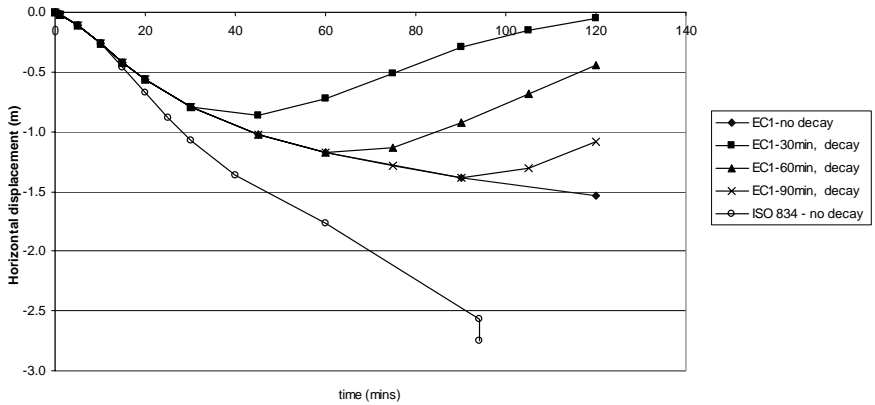


Fig 9. Horizontal displacements for an 8 m cantilever wall subjected to EC1 external fire curve.

Propped cantilever walls

The propped cantilever walls have very different behaviour. As for the cantilever walls, heating of the wall causes thermal bowing with the convex shape on the fire side. Because the tops of the walls are prevented from moving away from the fire, plastic hinges form at the base of the walls, with compressive stresses on the fire side and tensile stresses in the reinforcing steel.

Figure 10 shows the distribution of concrete stresses through the base of the wall. Steel stresses are not shown. Fig 11 shows horizontal displacements at mid-height of walls of various heights, during exposure to the ISO-834 fire. The 6 m high wall exhibits a displacement of 170 mm after two hours exposure, with no collapse, whereas the taller walls fail. The 12 m high wall fails after 30 minutes with a displacement of over 350 mm. Failure is the result of buckling caused by rapidly increasing deflections and reduced material strength due to elevated temperatures. The plastic hinge at the base of the wall occurs early in the fire. Failure by buckling occurs when a plastic hinge forms near mid-height of the wall, so that the central part of the wall moves inwards and the top drops vertically.

Behaviour of propped cantilever walls exposed to more realistic fires is illustrated in Fig 12. This shows that 10 m high walls, which fail after 35 minutes exposed to the ISO-834 fire, survive much longer under the EC1 fire exposure of 30, 60 or 90 minutes with a decay phase.

The horizontal reaction at the top of the same walls is shown in Fig 13. The maximum value of the reaction occurs when a plastic hinge forms at the base of the wall. The flexural capacity of the hinge, hence the horizontal reaction, will increase significantly if the wall is thicker or if there are two layers of reinforcing steel.

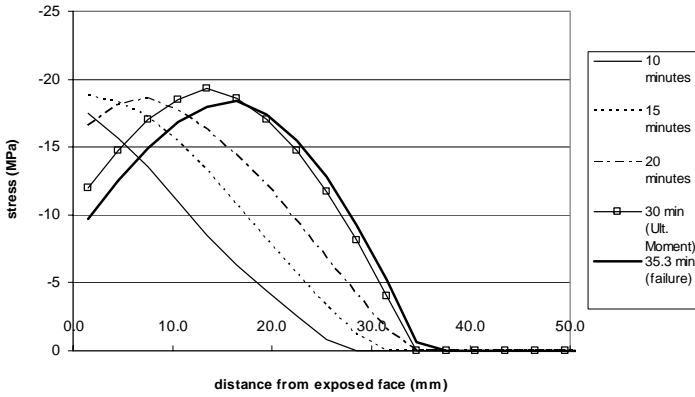


Fig 10. Concrete stress profile at the base of a 10 m propped cantilever wall exposed to the ISO 834 standard fire.

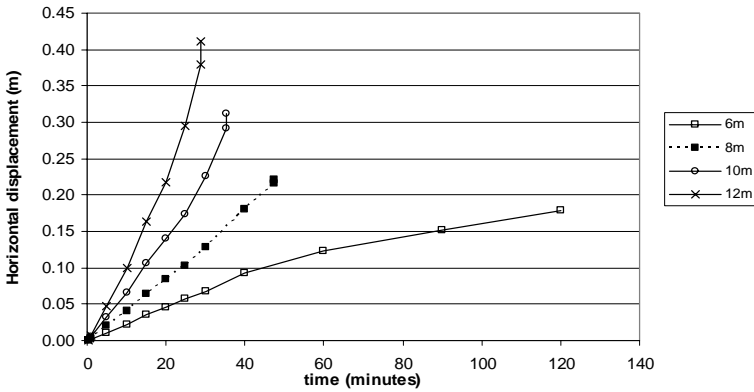


Fig 11. Horizontal displacements at mid-height of propped cantilever walls of different heights exposed to the ISO 834 fire.

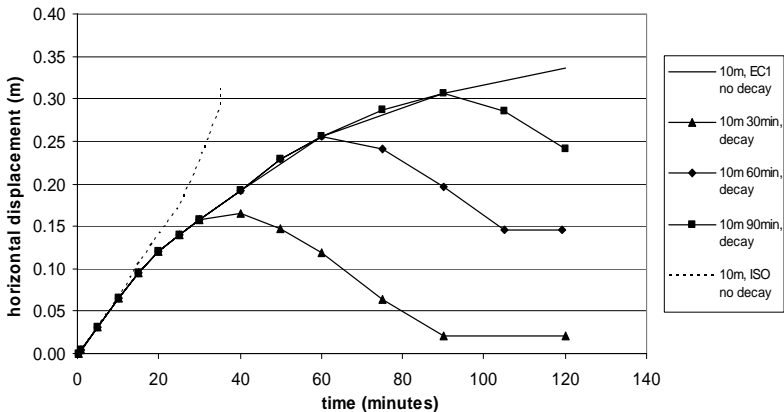


Fig 12. Mid-height displacement of a 10 m propped cantilever wall, for the EC1 external fire curve.

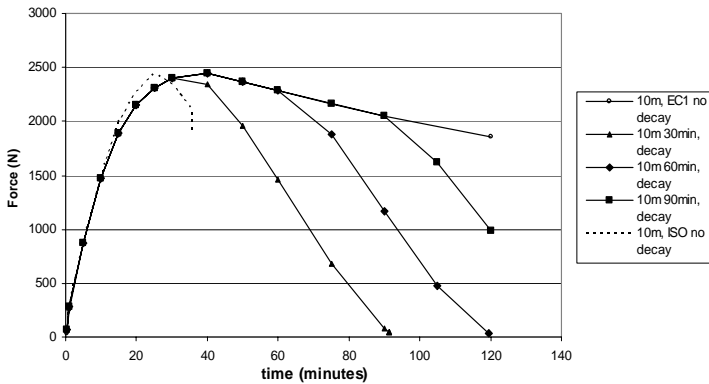


Fig 13. Horizontal reaction at the top of a 10 m high propped cantilever wall, for EC1 external fire curve.

Frames

Frames with varying degrees of restraint, partially exposed to fire, were investigated by analysing the frame shown in Fig 14. It can be seen that the fire occupies only half of the building, to simulate a migrating fire. The external spring at the right hand side represents the stiffness of the roof bracing or roof diaphragm which carries transverse loads back to the end walls.

Unbraced frames were analysed by giving the spring zero stiffness, and by providing pinned connections between the steel beams and columns.

Fully braced frames were analysed by giving the spring infinite stiffness, resulting in the propped cantilever condition. Partially braced frames were analysed by providing moment-resisting connections between the beams and columns and providing a spring of intermediate stiffness.

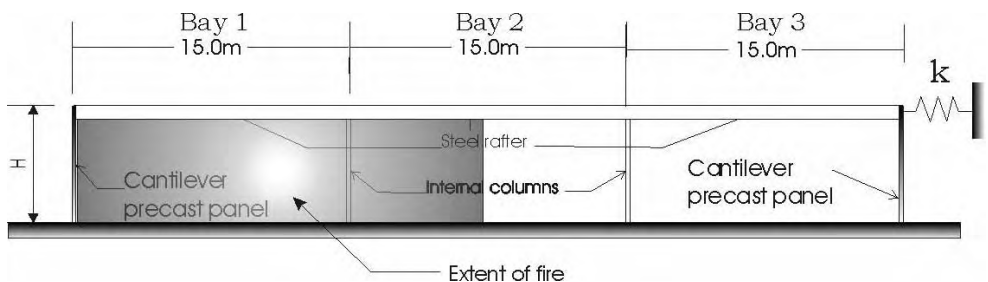


Fig14: Structural configuration of Frame A and the extent of the migrating fire.

Unbraced frames

Very few unbraced frames were analysed because they proved to be very unstable and difficult to model. Failure was sideways collapse after the flexural resistance at the base of the wall panels was exceeded.

Partially braced frames

The behaviour of a partially braced frame exposed to a migrating fire depends on the relative strength and stiffness of the heated wall panel which is trying to pull the building outwards, and the strength and stiffness of the beams and columns and roof bracing, all of

which are resisting outwards collapse of the heated wall. For the scenarios in this study, it was found that tall buildings with slender walls tend to collapse outwards, whereas less slender walls tend to be pulled inwards by the collapsing rafter during a fire.

CONCLUSIONS AND RECOMMENDATIONS

This research project was conducted to analyse the behaviour of slender cantilever concrete wall panels in industrial buildings. The industrial buildings comprise steel rafters supported on internal steel columns and load-bearing precast panels with no columns at the perimeter of the building. The steel frame is not fire protected. The precast concrete panels are cantilevered at the base and do not have columns attached.

The analysis was conducted using SAFIR, a non-linear finite element program. The scope of the analysis covered the behaviour of free-standing cantilever walls, propped cantilever walls attached to a fire resistant roof structure, and cantilever walls attached to a non fire protected steel frame.

Free standing cantilever walls

The behaviour of free-standing cantilever concrete walls is very sensitive to the slenderness ratio. Walls with high slenderness ratios experience very large deflections when exposed to a fire on one side. The deflections, due to thermal bowing and enhanced by P-delta effects, lead to outward collapse if the walls do not have sufficient flexural strength at the base, or if the foundations do not have sufficient resistance to the overturning moment.

If the wall panels cannot be effectively connected to the steel frame, then measures have to be taken to control the thermal bowing deflections including provision of: intermediate concrete columns fixed to the wall panels, increasing the thickness of the wall panels, or increasing the quantity of reinforcement in the wall panels.

Propped cantilever walls

A wall behaves as a propped cantilever if the top edge is attached to a steel frame which is fully fire-protected. Propped cantilever walls do not experience large out-of-plane deflections when subjected to a fire on one side. They bow inwards towards the fire and form a plastic hinge at the base. After a plastic hinge has formed at the base of the walls, accompanied by significant cracking, further deformation may cause the walls to buckle under their own weight. Slender walls exhibit larger out-of plane deflections and shorter survival times compared to stockier walls. Stockier walls, however, impose larger horizontal forces on the supported rafter compared to the more slender walls.

Braced frames

Braced frames are those relying on bracing such as the roof diaphragm for lateral load resistance. Behaviour of the walls is very similar to the propped cantilever case, except that internal collapse is possible. If the frames are fully braced to prevent outwards movement, the walls will collapse inward without buckling of the wall panels, even with slenderness ratios as high as 100. The fire causes plastic hinges to form in the steel rafter, and when the rafter collapses inwards, the attached wall panel is pulled inwards. This prevents outward collapse of the wall panels onto the neighbouring property and also prevents a buckling failure of the wall panels.

Unbraced frames

Unbraced steel frames incorporating concrete wall panels perform very poorly when the wall on one side is exposed to a fire. The sway of unbraced frames during the fire is resisted only by the flexural strength at the base of the cantilever wall panels. When the sway of the frame has produced sufficient overturning moment to form a plastic hinge at the base of the walls, the frame loses its sway resistance and collapses over onto the neighbouring property, even for wall heights as low as 6 metres.

Partially braced frames

For the partially braced frames analysed in this study, it is assumed that the fire does not occupy the whole building, so that sway is resisted by frame action and the flexural strength of the cantilever walls in the non-fire-affected part of the building. If the height of the wall exceeds 9 metres and its slenderness ratio exceeds 65, the frame will sway during a fire and collapse on to the neighbouring property when the overturning moment exceeds the sway resistance of the frame. This occurs when plastic hinges form in the unheated columns, leading to the loss of sway resistance. Walls with heights less than 8 metres and slenderness ratios under 65 collapse inwards during a fire when plastic hinges form in the rafter, pulling the wall panels in.

Connection of the wall panels to the steel frames

The wall panels should always be connected to the steel frames to prevent outward collapse of the wall panels following thermal bowing. This recommendation applies whether or not the steel frames are fire protected. To prevent the outward collapse of the panels, strong and well designed connections between the panels and the frame are required. The connections may have to withstand very high pull-out forces while exposed to high temperatures. Epoxied connections should not be used because they have very poor fire resistance.

If a wall consists of a series of panels side by side, an eaves tie is required to tie all the wall panels together, hence preventing individual panels from deforming and collapsing outwards during a fire. This is particularly important if there is not a steel rafter attached to every wall panel. The Building Code of Australia [3] provides guidance on connection details for precast concrete wall panels.

Building design

The design of wall panels should follow the slenderness limits recommended above, depending on the level of bracing available during a fire. Roof cladding should not be relied on for diaphragm action if the cladding is aluminium, or if there are light timber purlins or very large skylights. There is generally no significant advantage in providing applied fire protection or over-designing the steel rafters because although such measures can delay the collapse of the frame, the time delay increases the probability of exterior collapse caused by buckling of slender wall panels.

FUTURE RESEARCH

It is recommended that future research should include:

- Analysis with real fire curves and different types of concrete.
- Three dimensional analyses of whole buildings exposed to migrating fires.
- Fire resistance of the wall to rafter connections and wall to eaves tie connections.
- Analysis of base connections of cantilever walls to determine their ability to sustain the moments due to the thermal bowing deflections.

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