ASPECTS OF THE BEHAVIOUR OF SINGLE STOREY INDUSTRIAL BUILDINGS IN FIRE

A THESIS SUBMITTED BY

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FOREWORD

This thesis contains no material which has previously been submitted for an award or degree at any University. To my knowledge, the work reported in this thesis is original and contains no material published by other investigations, except where appropriate reference has been given to the source of the material.

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ANTHONY BORTOLI

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SYNOPSIS

In Australia, single storey industrial buildings are a common form of construction; with fires within these buildings also being relatively common. The behavioural response of such buildings in real fire conditions is investigated; more specifically the behaviour of the connections between the structural concrete and steel components of portal framed industrial buildings. The research presented in this thesis will provide an informative background as well as useful data regarding the response of certain connection types in real fire conditions for single storey portal framed industrial buildings.

Chapter 1 covers the background of the research topic in relation to current forms of construction and associated regulatory requirements for constructing such buildings. Chapter 2 considers the relative regulatory requirements as set down by the BCA (Building Code of Australia) in greater detail and gives a detailed description of current warehouse construction practices; with various design approaches highlighted and compared. Chapter 3 investigates the behavioural response of such buildings via the use of a numerical model to study the deformation characteristics of concrete walls in fire. In Chapter 4 the model is used for both vertical and horizontal walls exposed to various heating scenarios. Chapter 5 studies the behaviour of two types of connection, common to today's industrial buildings, under elevated temperature conditions. A rationalised discussion is then presented in Chapter 6 with major conclusions highlighted as well as recommendations for future work, based on the results of this investigation, also given.

DEFINITIONS

AUBRCC	Australian Uniform Building Regulations Coordinating Council
BCA	Building Code of Australia, 1990.
BHP	Broken Hill Proprietary
Connections	the means of attaching a wall panel to a *supporting member* and the *supporting structure*.
Det	Determinant
End Walls	external wall panels that are laterally supported by roof bracing or purlins.
FRL	Fire Resistance Level as defined and required by the BCA.
Horizontal Panel	a panel which resists lateral wind loads predominantly through bending action between supporting columns.
Lateral Supporting System	the combination of connections and *supporting member* (if relevant) which attach the panels to the *supporting structure*.
Loadbearing	means intended to resist vertical forces additional to those due to its own weight.
P-Delta effects	action effects resulting from the lateral displacement of the wall panel or structure.
Precast Panel	a concrete element that is cast and cured in other than its final position.
Rafter	a steel beam spanning to a load bearing wall panel or connected to columns so as to form a loadbearing frame.
SFT	Standard Fire Test
Side Walls	the walls between which the main rafters or frames span.
Supporting Member	a member forming part of a *lateral supporting system* which provides lateral support to panels.
Supporting Structure	the building or roof (in the case of a loadbearing panel) to which the panel is attached by the lateral support to panels.

Tilt-up Panel	a precast element normally cast on-site in a horizontal position adjacent to its final location for lifting into position.
Vertical Panel	a panel which resists lateral wind loads predominantly through bending action over the height of the storey.

NOTATIONS

ε	strain; (mm/mm)
σ	stress; (kN/m^2)
Δ	total lateral displacement at top of wall - from analysis; (mm)
σ / f_{ay}	proportional yield stress of steel; (kN/m^2)
σ / f_c	proportional yield stress of concrete; (kN/m^2)
δ_{lr}	limiting value of relative lateral displacement; (mm)
f'_c	compressive cylinder strength of concrete at 28 days (kN/m^2)
f_{ay}(20 °C)	design value for the yield point of steel for normal conditions of use; (kN/m^2)
H'	the height of the concrete panel considered for analysis; (m)
H_c	the height of the column from the base of the wall to the uppermost connection between panel and supporting member or lateral supporting system; (m)
H_f	the vertical distance between connections on a horizontal panel; (m)
H_w	the height of a wall for a single storey building; (m)
M_b	the ultimate moment capacity at the base of a vertical panel; (kN-m per m)
p_w	reinforcement ratio; (%)
t_w	the thickness of wall panel for a single storey building; (m)

Subscripts

c	concrete
h	horizontal
i	internal
lr	limiting relative value
o	original
s	steel
v	vertical
w	wall

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CHAPTER 1 INTRODUCTION

Single storey warehouse buildings are a common form of construction. These buildings often house large quantities of combustible materials, are rarely fitted with sprinklers, and often house operations which are hazardous with respect to fire initiation.

It is not suprising, therefore, that fires in such buildings are a frequent occurrence relative to the frequency of fires in other commercial buildings. To date, there has been no evidence of death resulting from fires in such buildings. This is probably attributable to the fact that most fires appear to develop after hours when there are no or few occupants still present within the building.

Fires in these buildings are of concern to fire brigades as these fires tend to be very large. Extinguishment of the fire is generally not possible and the role of the brigades is to assist with preventing fire spread to the adjacent properties. This is usually done by playing water on the walls of the adjacent buildings. Other important factors relevant to limiting fire spread, are the presence of fire-resistant external walls and/or the spacing between the adjacent buildings.

The Building Code of Australia -BCA (1996), gives the fire-safety objectives of the regulations. These are summarised as follows:

(i) to provide an adequate level of safety for the occupants of the building and the fire fighters;

(ii) to provide an adequate level of protection for adjacent properties.

Traditionally ~~single storey~~ warehouse buildings have been constructed using steel frames and masonry external walls. This was particularly the case where the external walls were required to be fire-resistant when located closer than 3 m from the site boundary. Otherwise, the building was clad with steel sheeting. Whilst in a large fire, the masonry walls eventually crumbled, they were considered to provide an adequate level of fire separation. Although the external walls within a certain distance of the site boundary were required to have a fire-resistance level, the columns and rafters were permitted to be unprotected and not required to achieve the level of fire-resistance required for the walls. This situation is essentially the same today.

Changes in building fashion, technology, and construction economics have resulted in the external walls of warehouse buildings being most commonly constructed using concrete panels. This is sometimes the case even when a fire-resistant wall is not required.

The use of such concrete panels for the external walls of buildings has raised the question of whether this form of construction can satisfy the above-mentioned objectives. To be specific, will the panels become detached and fall outwards thus potentially endangering the life of fire fighters outside the building and adjacent property? Are there special design considerations that should apply to ensure that the above objectives can be achieved?

In this regard, the Building Code of Australia (1990), specifies the following:

C1.11 Performance of external walls in fire

(a) *If a building having a rise in storeys of not more than 2 has concrete walls that could collapse as complete panels (eg tilt-up and precast concrete), they must be designed so that in the event of fire the likelihood of outwards collapse of the panels is minimised.*

(b) *Compliance with Specification C1.11 satisfies (a).*

Amendment 7, Building Code of Australia

The above issues are considered

~~Chapter 2 considers the above issues in more detail~~, and ~~gives~~ a detailed description of current warehouse construction. The behavioural characteristics of these buildings are described using a number of case studies, and the literature on the subject is reviewed.

As might be expected, various design approaches have been proposed to ensure adequate behaviour in fire. The approaches are summarised and reviewed in Chapter 2.

The occurrence of a fire within a warehouse building will result in heating of the inside faces of the external wall panels, as well as the unprotected steel frame. Heating of the inside of the panel will cause it to deflect substantially - and it is possible that the deflection of the panel may be in a different direction to that of the frames to which the walls are attached. To study the deformation behaviour of walls, a numerical model was developed by the author. This model is described in Chapter 3.

In Chapter 4 the model is used to study the behaviour of isolated vertical and horizontal panels under various heating regimes. The model is then used to evaluate the critical aspect of the design philosophy developed by O'Meagher et al. (1991).

Not only are the panels and the frame heating up during a fire but also the connections. It is important, therefore, for the behaviour of connections under elevated temperature conditions to be determined. This is the purpose of the experimental work presented Chapter 5.

It is intended that the work presented in this thesis will provide background and data relevant to the current debate on the behaviour of industrial buildings in fire.

CHAPTER 2 BEHAVIOUR OF SINGLE-STOREY INDUSTRIAL BUILDINGS IN FIRE - A REVIEW

2.1 Introduction

The purpose of this chapter is to summarise and review the literature related to the behaviour and design of single-storey industrial buildings in fire.

In recent years a number of researchers, particularly in Australia, have attempted to understand various behavioural aspects of these buildings in fire and various papers and reports have been written that deal with these aspects as well as the principles for designing these structures.

The interest in the behaviour of these buildings has been largely generated from the debate about whether it is necessary for the steel frames to be fire-protected in situations where the frames provide lateral support to external wall panels, and from a concern about large concrete panels falling outwards in the event of fire.

As noted in chapter 1, Clause C1.11 of Specification C1.1 of the BCA requires buildings with concrete or reinforced masonry external walls to be designed, such that in the event of a fire, the likelihood of outwards collapse of the wall is minimised.

According to Part A3 and Table C1.1 of the BCA, the single storey industrial buildings, which form the subject of this thesis, are classified as Class 7, Type C construction. According to Table 5, a building or compartment having an area of less than 3000 m^2 must be surrounded by walls having a fire-resistance level of 90 minutes if the walls are located closer than 3m from the fire-source feature (normally the site boundary).

Two concessions are given in the BCA for these buildings.

Spec. C1.1, 2.5 (a)

(i) ***steel columns*** - *except in a fire wall or common wall, a steel column need not have an FRL in a building that contains only one storey.*

In various situations, the use of unprotected steel roofing members is allowed.

Spec. C1.1, 3.5

(ii) ***Roof*** - *A roof need not comply with Table 3 if its covering is non-combustible and the building,*

(a) has a sprinkler system installed throughout; or

(b) has a rise in storeys of 3 or less; or

(c) is of Class 2 or 3; or

(d) has an effective height of not more than 25m and the ceiling immediately below the roof has a resistance to the incipient spread of fire to the roof space of not less than 60 minutes.

For a typical situation, (see Figure 2.1), the BCA requires the concrete wall panels to have an FRL of 90/90/90. These numbers specify the performance requirements with respect to structural adequacy, integrity and insulation. For a wall panel, integrity and insulation are essentially a function of thickness and will easily be achieved provided the panel has sufficient thickness. However it is important to note that typical wall panels will not achieve an FRL of 90 minutes with respect to structural adequacy, unless they are laterally supported at the top (see discussion in Chapter 4).

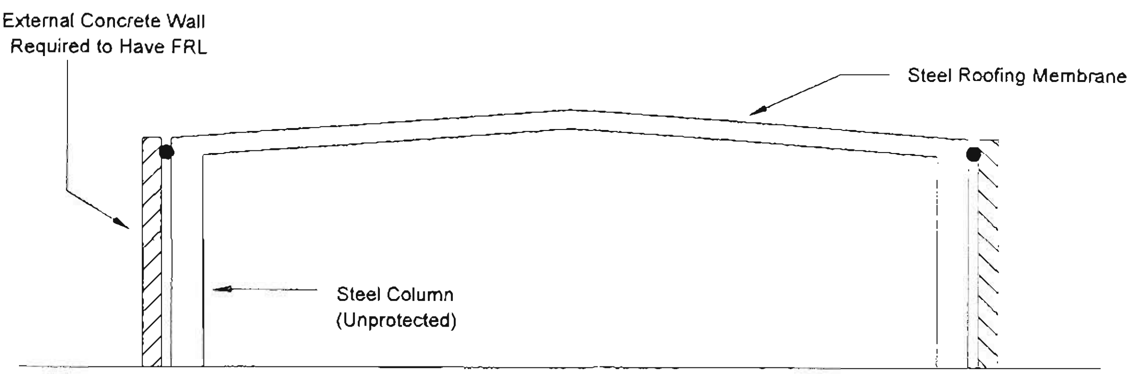


Figure 2.1 BCA Concession for Steel Columns

The BCA also stipulates, in Specification C1.1 - Clause 2.2, that members offering lateral support to elements required to have a Fire Resistance Level (FRL), should have the same FRL. Therefore, as the walls must be laterally supported at the top, Clause 2.2 strictly requires the frames to be protected to achieve an FRL of 90 minutes. However, in Australia, there are no situations to date where this has been insisted upon - essentially because in the past, protection of the frames has never been required. This *"support-of-another part"* requirement, with respect to lateral support, has been deleted or amended in Queensland, Victoria and South Australia so as not to apply to these buildings.

The above dilemma, of whether to protect the steel frames or not, resulted in the initiation of an Australian Uniform Building Regulations Coordinating Council (AUBRCC) project in 1989. This project was undertaken by BHP - Melbourne Research Laboratories and attempted to address some of the above issues. A report by O'Meagher et al. (1990) was issued in 1990. The report argued that fire protection of the steel frames was not required to satisfy the regulatory objectives of the BCA. It was argued that it is not necessary for wall panels to remain essentially vertical (as would be required in a fire test) to provide adequate separation provided they remained attached to the steel frame as it deformed. This is illustrated in Figure 2.2.

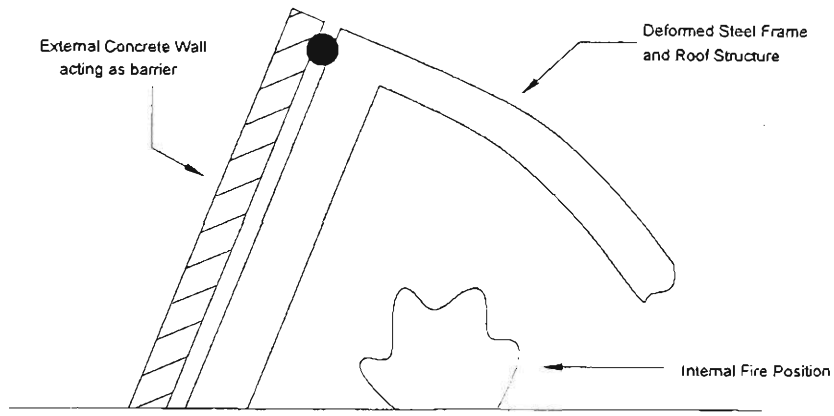


Figure 2.2 Possible Deformed Shape of Single-Storey Industrial Building

Effective compartmentation is also a regulatory requirement under the BCA. As to what constitutes effective compartmentation, for single-storey buildings, the BCA states that if an external wall is located up to 3m away from a fire source feature then no provision for an FRL is required. From this it is considered that if the external wall provides at least this level of fire separation then effective compartmentation has been satisfied.

In some cases where the external concrete wall panels deform inwards, to the extent of almost becoming horizontal, the end result is a formation of a greater barrier to fire than a non-fire resistant wall located 3m from the boundary. This boundary exists regardless of the time at which large deformations or collapse occurs, so long as the wall remains effectively intact (O'Meagher et al., 1990).

It was recognised that such gross deformation may result in gaps opening up between panels. This strictly constitutes failure of the wall with respect to integrity[1] - however the panels, even in this state are considered to provide a substantial and sufficient barrier to radiation and flame. Even if large gaps do form, then the concrete wall is still effective in acting as a fire barrier - more so than that of a non-fire resistant wall located 3m from the boundary. See also Part C3 Clause C3.1(a)(ii).

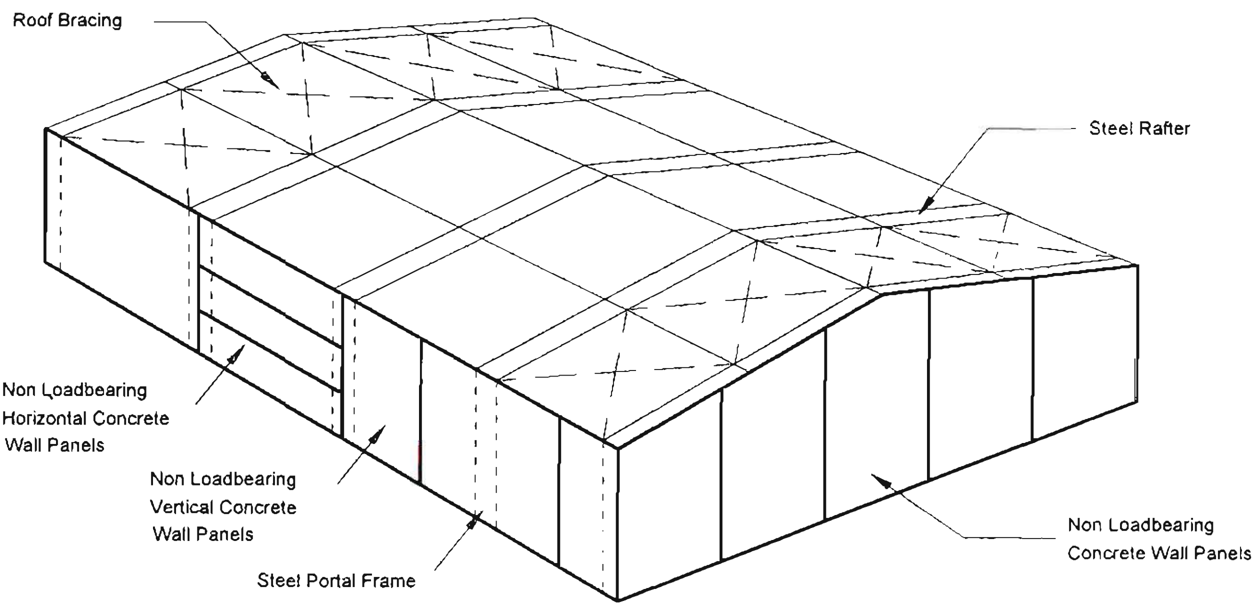
The importance of adequate connections between the panels and the supporting frame was also noted by O'Meagher et al. (1990).

1 Defined in AS 1530.4 as the development of cracks and fissures which allow the passage of flame and hot gases to pass.

The conclusions of the above report were accepted by AUBRCC and changes to Clause C2.2 are currently being considered to bring regulation uniformity across the States.

2.2 Single-Storey Warehouse Buildings

In the case of single-storey industrial buildings incorporating steel portal frames, concrete cladding panels can be either horizontal or vertical (see Figure 2.3).



NOTE: The Non-Loadbearing Concrete Wall Panels are Either Pinned or Fixed at the Base

Figure 2.3 A Single-Storey Non-Loadbearing Portal Frame Industrial Building

In both cases, the panels are considered by the BCA to be non-loadbearing, with respect to gravity loads, as they support self-weight and carry no forces from the roof structure. They are required however to resist lateral wind loads. Horizontal panels are generally less than 2m in width and resist wind loads by spanning between supporting columns and are generally supported with steel clips as shown in Figure 2.4.

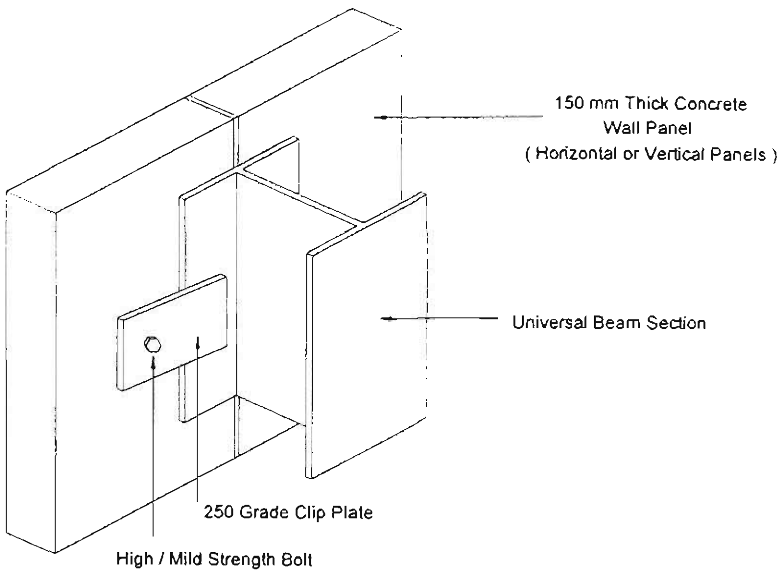


Figure 2.4 Typical Steel Clip Plate Bearing Connection

Vertical panels, on the other hand, span between the foundation and the roof and may be attached by a variety of systems to the steel frames. There are a variety of different vertical panels, and frequently an eaves tie member is provided to support adjacent panels under wind load conditions (see Figure 2.5) .

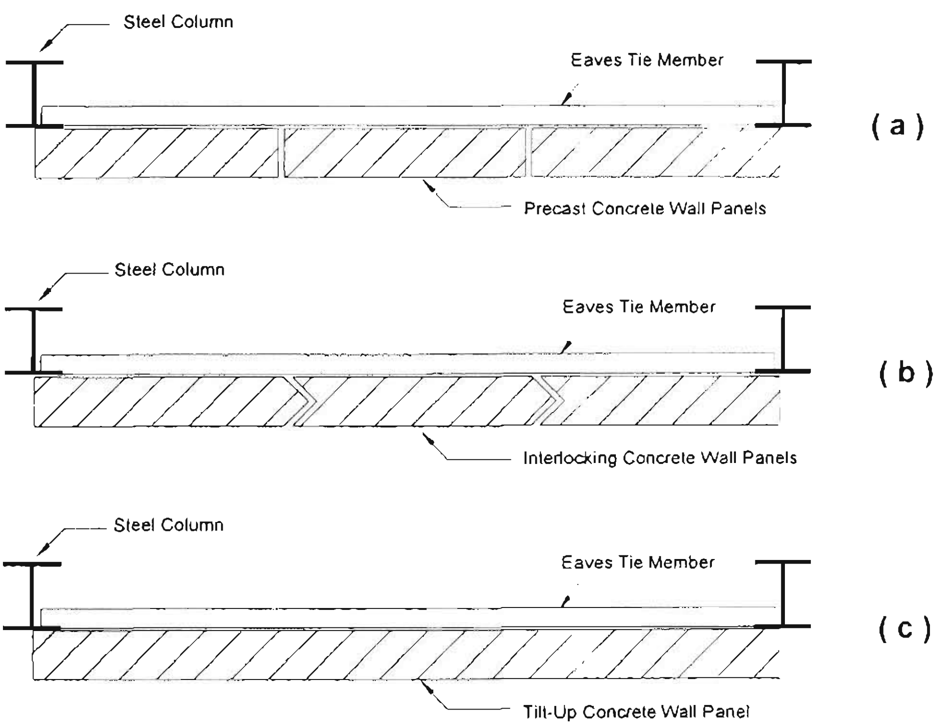


Figure 2.5 Schematic View of Various Vertical Panel Lateral Support Systems

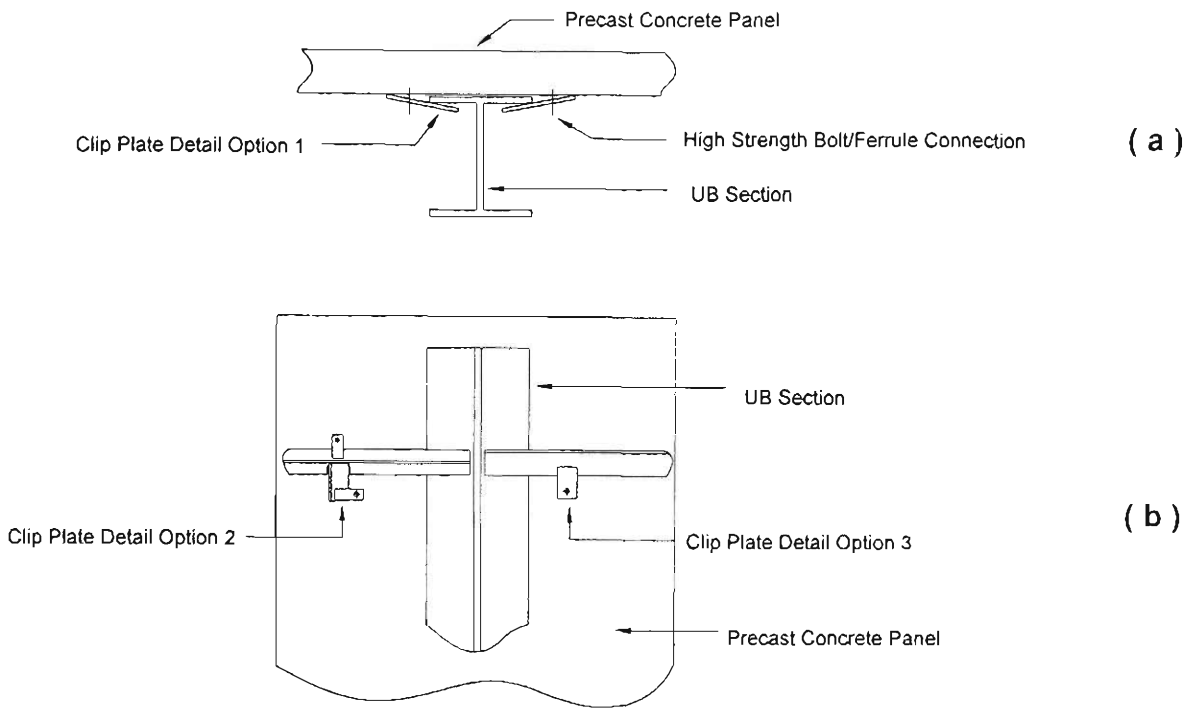


Figure 2.6 Methods of Connection for Support in Fire

In the past, most designers have designed and detailed connections with panel erection and wind forces in mind. The importance of connections in the fire situation has become of critical importance although in the past it was rarely considered.

As will be noted from Figure 2.6, a typical connection of the type assumed to offer support in fire consists of steel clips connecting the panels directly to the column flanges or to an eaves tie member. In both cases, the clip plate is bolted to the concrete wall panel via a threaded insert or cast-in ferrule. The clip plate is often not welded to either the eaves tie member or the column flange.

In the case of tilt-up construction, the concrete wall panels are cast on-site and are erected once the slabs have cured. In contrast, precast panels are pre-fabricated off-site and then transported to the site where they are erected into position along the external faces of the portal frame. Ease of fabrication and transportation generally mean that precast concrete wall panels are smaller than tilt-up panels.

Once the floor slab is poured, the steel portal frame is able to be positioned and fixed into place. The steel frame is generally rectangular in shape with single spanning frames spaced at regular distances along the length of the building as shown in Figure 2.3. Bracing is provided in the roof to facilitate resistance to end wall wind loading and to provide overall stability.

In practice, the roof sheeting in association with the purlins, can provide substantial in-plane bracing at the roof level (Bryan, 1972). During the early stages of fire development it is likely that the roof will continue to provide significant restraint - especially away from the fire (O'Meagher et al., 1990).

Once the portal frame is erected the concrete panels are then positioned in place by connecting the base of the panel to the footing and the panels to the steel portal frame.

2.3 Building Behaviour in Fire

In the event of fire in a single-storey industrial building, there are a number of possible scenarios depending on the severity of the fire (O'Meagher et al., 1990 and Gnanakrishnan, 1990). The steel frames may undergo severe deformation; panels may deform significantly and become detached from the supporting frame; alternatively, relatively little damage may occur.

In this chapter the influences of fire on a single frame, on an isolated wall panel, and on the overall building are considered. The role of the connection between concrete panels and frame is also discussed.

2.3.1 The Developing Fire Concept

Real fires are highly variable and difficult to quantify. Factors that most affect the fire characteristics are as follows; (Malhotra, 1982):

- the quantity and orientation of combustibles stored within the building,
- the nature and geometry of the combustibles, and
- the degree of ventilation.

In a real fire situation, it is reasonable to assume that the fire develops slowly from the time of initiation, with various parts of the roof and walls being heated at different rates. As the fire escalates in size, so too does its' intensity. It is likely that the growing fire will heat the adjacent members differentially leading to variations in steel and air temperatures throughout the building. This scenario has been described in terms of a "developing fire concept" by O'Meagher et al. (1990).

The above authors consider that it is unlikely that a steel frame will be at a uniform high temperature but that parts of a fire-affected frame will be at different temperatures.

Other gradients in temperature are likely. For example, it is likely that while the fire is small, the highest air temperatures will be closest to the fire. As the fire further develops, the temperature close to the roof will increase. However, substantial venting of the flames through the roof will probably result in a reduction of air temperature near the roof members.

2.3.2 Frame Behaviour

The behaviour of a steel frame in fire is now considered.

Much can be learnt from studying the geometry of buildings after the occurrence of fire, and post-fire observations associated with a number of large fires in steel-framed industrial buildings, are reported in Appendix A. Some of the observations have been reported in the literature whilst others are the result of investigatory work undertaken as part of this research project.

Observations of post-fire industrial buildings show the steel frame structure having deformed and distorted away from the external concrete wall panels. Due to the heat transfer effects of the internal fire, the steel roof members have a tendency to feel the effects of the fire prior to other structural components within the building and tend to warp and collapse under their own self-weight. The frame then collapses inwards as the rafters sag and collapse forcing the steel columns to follow this deformation due to the rigid connection that exists between the column and the rafter at the haunch.

A number of publications have addressed the behaviour of the frames in fire and these are now considered.

Constrado - [1980]

In 1980, Constrado published a document entitled *"The Behaviour of Steel Portal Frames in Boundary Conditions"*. At this time the building regulatory authorities in the United Kingdom were suggesting that perhaps it was necessary to fire protect the entire portal frame assembly, as opposed to just the columns. This document was prepared in response to this issue.

In the United Kingdom, it is very rare for portal frame buildings to have concrete panels as wall cladding. Due to the climate, specific thermal insulation requirements must be achieved and walls are either of insulated steel or masonry construction. Therefore this publication does not consider the interaction between concrete wall panels and frames.

Regulations in the United Kingdom, require only the column to be protected. This has always been the case.

The above publication considered the behaviour of frames in fire from a "common sense" engineering approach.

According to this publication, as the fire escalates in growth and intensity, the members within the roofing membrane are affected the most. As the temperatures increase within this upper region of the building, the strength and stiffness of the structural steelwork are reduced. The steel rafters tend to heat up and expand which initially causes an outward movement of the eaves. This results in the formation of plastic hinges close to the eaves as the bending moments at the eaves increase. The combination of increased bending moments and axial force in the rafter (see Figure 2.7) was recognised as leading to flexural - torsional buckling such that the rafter may end up with its web in a horizontal position.

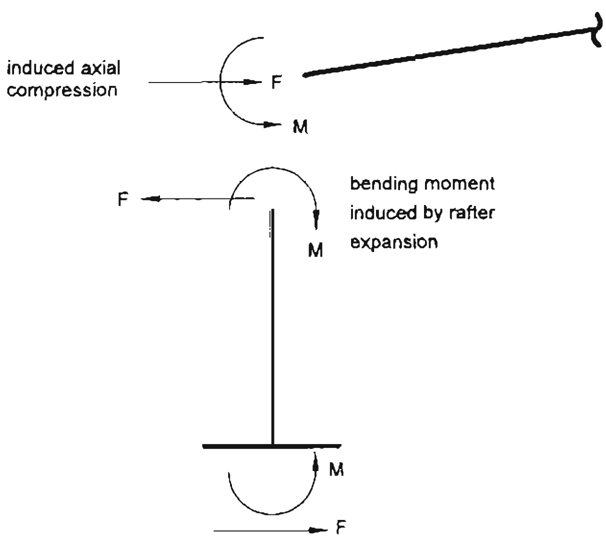


Figure 2.7 Forces and Moments Acting on the Portal Frame Members - Free Body Diagram

As the rafter undergoes gross deformation downwards, the columns will be pulled inwards.

The above publication suggests that adequate frame behaviour will be achieved if the base connections of the frame have sufficient moment capacity. The required capacity was determined as follows:

(i) observation of some actual fires in warehouses suggested that typical deformations were such that the columns rotated inwards by $\approx 1°$.

(ii) assuming this displaced shape, and the position of the point of contraflexure within the rafter, the bending moment at the base required for equilibrium was determined.

(iii) the base connection, which is protected because the column is protected, must be designed to achieve this capacity. In practice this is relatively easy to achieve. "Pinned connections" are not recommended.

The above approach is based on the assumption of a symmetrical fire and of a certain frame geometry. In Australia, at least, column rotations - even when columns have been protected - have often been found to exceed the assumed 1°. Column rotations of more than 15° have been obtained (see Appendix A). The final geometry of the frame is dependent on the severity of the fire.

The Constrado publication also recognises that frames that have collapsed inwards (ie. the frames nearest the point of fire origin) will provide some restraint to the remainder of the steel structure, thus minimising the chances of an outwards overall collapse.

Gnanakrishnan - [1993]

The report prepared by Gnanakrishnan, (1993), entitled *"Likely Fire Performance of One and Two Storey Buildings with Precast Concrete or Tilt-up Panels or Precast Concrete Cladding"*, does not consider frame behaviour in a quantitative manner but makes a number of general post-fire observations.

Gnanakrishnan analysed the behaviour of various structural components subject to fire but did not attempt to analyse the entire building structure subject to these conditions.

Regarding the behaviour of the steel frame in fire, Gnanakrishnan gives a brief description of the stages of collapse of a steel frame based on post-fire observations.

The development of a typical industrial building fire was considered qualitatively with respect to the likely behaviour of the entire building subject to various fire exposures. The fire scenarios considered were:

(i) internal fire adjacent to an external wall

(ii) internal fire adjacent to an internal wall

(iii) internal fire at the corner of a building

(iv) internal fire, (not reaching the roof), on the ground floor

(v) internal fire spread throughout the building

(vi) external fire near a wall

Gnanakrishnan concludes that intensity, spread and duration of the fire in a building are dependent on the initial fire source, availability of fuel, building size, building openings and ventilation. The problem of analysing the fire behaviour of the entire building structure was recognised as being too difficult due to the large number of contributing factors.

Gnanakrishnan observes that modern industrial buildings of this type often contain a vast variety of stored goods; varying in both quantities and volatility and that it is because of this that high temperatures can occur within these buildings. The combination of severe fires and unprotected steel is considered to give rise to substantial deformations of the frames.

Gnanakrishnan describes the behaviour of the steel column / concrete wall panel interface as follows:

"Through the effects of the increased fire load, present within the building, the steel column has a tendency to expand and bend slightly; ie. bow inwards. This is due to partial shielding of the column flange abutting the wall panel. The concrete's low conductivity causes the panels to bow outwards and exert considerable force on the connections which would have already suffered a

loss in strength due to increased fire exposure. These additional forces would cause the connections to fail and would result in the wall cladding collapsing."

O'Meagher et al. - [1990]

In 1990, a report was published by BHP Research - Melbourne Laboratories (O'Meagher et al.) which addressed the behaviour of these buildings. In a later paper, O'Meagher et al. (1992) explained and expanded the contents of the above report. The investigation described in these publications was undertaken to determine whether steel frames have a tendency to collapse "inwards" or "outwards" in the event of a fire. These two possibilities are shown schematically in Figure 2.8 (a) and 2.8 (b).

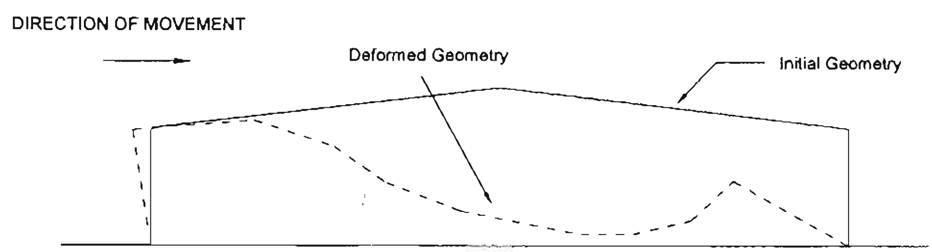


Figure 2.8 (a) Acceptable Inwards Collapse of Portal Frame

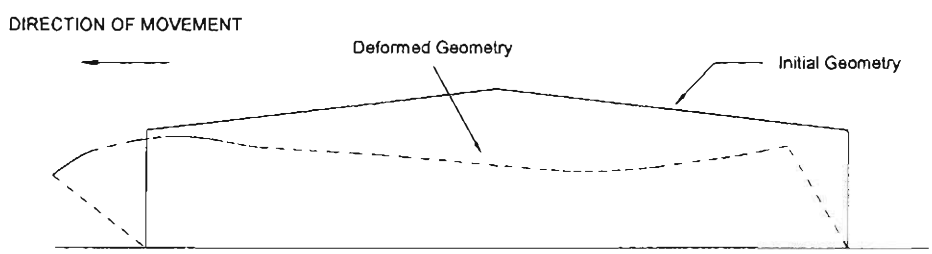


Figure 2.8 (b) Unacceptable Outwards Collapse of Portal Frame

The above approach involved analysis of single-storey industrial buildings incorporating steel portal frames. Investigation of both the steel portal frame and the attached cantilevered concrete walls in fire was undertaken to provide an overall understanding of behaviour for single-storey industrial buildings given various fire scenarios.

Some of the key factors that were taken into account were:

(i) the effect of elevated temperature on steel properties,

(ii) the effect of differential heating,

(iii) the effect of second-order displacements on frame behaviour,

(iv) the presence of a lateral in-plane restraint to the frame from the roof,

(v) the restraint provided by the column base connections, and

(vi) loads due to the attached external concrete walls and the portal roof structure.

The frame geometries adopted for the analysis were 15 m and 20 m span portals, each having a height of 6.5m. Two different steel section sizes were used for each of the above geometries. Various heating scenarios were considered by imposing a number of distributions of temperature around the frame as shown in Figure 2.9.

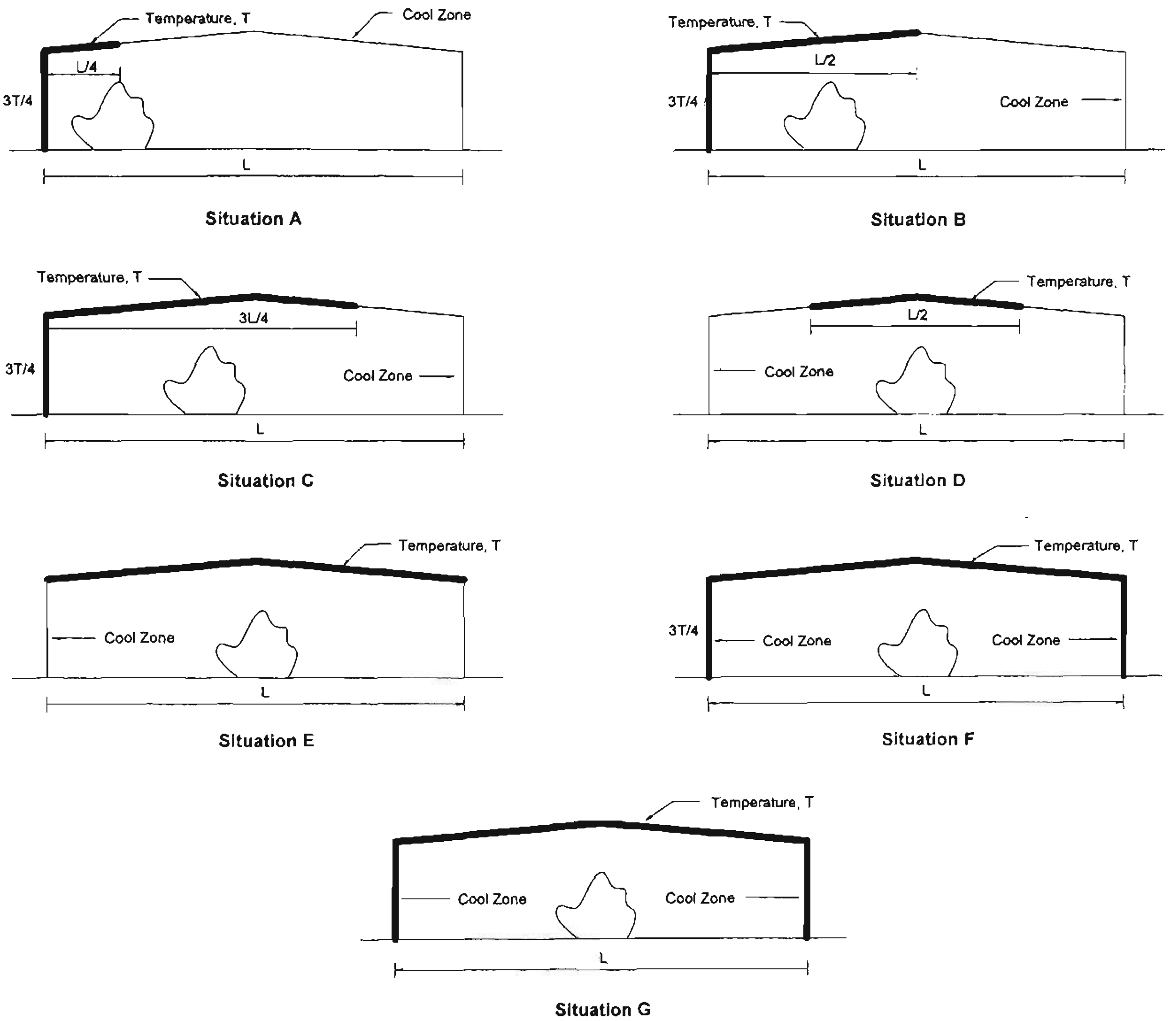


Figure 2.9 Heating Situations Considered by O'Meagher et al. (1990)

The frames were analysed using the finite element program ABAQUS with the frame temperature being increased until very large displacements occurred.

As a result of the above analyses, it was found that plastic hinges (regions of extensive plasticity) were formed at or near one end of the rafter and within the rafter close to the end of the heated region. These hinges are caused by the yielding of the rafter as the yield stress of the steel falls with an increase in temperature and as the bending moments due to thermal effects increase.

The analyses results showed that the desired frame deformation (see Figure 2.8 (a)) will, in practice, be achieved. This was found to be true irrespective of whether the frame supports concrete wall panels, or whether the columns are protected. For frames with "pinned" base connections, desirable frame deformations ("inwards" deformations) were obtained for all unsymmetrical frame temperature distributions. In practice, unsymmetrical frame temperature distributions will always occur. However, even for frames with uniform temperature distribution, the desirable deformations occurred provided a small degree of moment resistance was present at the base connections. The level of resistance required will be easily achieved by practical base details.

That desirable frame geometries are obtained in real fires is supported by the findings presented in Appendix A.

It was also recognised that as the fire develops within a building, and as the frames progressively deform, the frames will act as "anchors" to the rest of the building.

√ YES

2.3.3 Wall Behaviour

In this section, the behaviour of wall panels under fire conditions is considered.

As can be seen from the case studies given in Appendix A, concrete walls subject to an internal fire will tend to deflect outwards, provided they are effectively fixed at the base. This is principally due to differential thermal expansion of the wall as illustrated in Figure 2.10; (O'Meagher et al., 1990 and Gnanakrishnan, 1993).

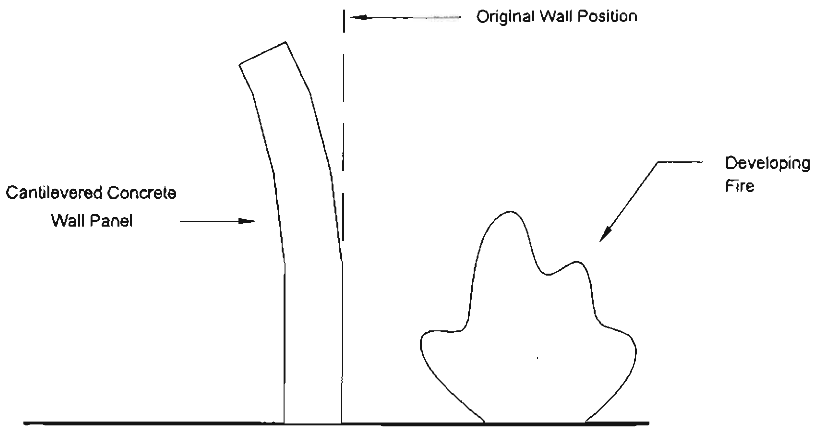


Figure 2.10 Deformation of a Cantilevered Wall

If the vertical wall panel is dowelled at the bottom, and supported laterally at the top, it will behave as if it is a cantilever due to prying against the column flange. This aspect of behaviour is discussed at some length in Chapter 4.

Constrado (1980), does not consider the behaviour of any attached concrete walls, as in the United Kingdom, walls are almost entirely insulated steel or masonry construction.

O'Meagher et al. (1991), have looked extensively at the behaviour of reinforced and prestressed concrete wall panels under fire conditions. An evaluation of isolated wall behaviour is presented in Chapter 3 using a variation of the method used by O'Meagher and Bennetts, (1991). Both vertical and horizontal panels are considered.

Gnanakrishnan (1993), considers that concrete wall panels may suddenly fall either inwards or outwards without any visible sign of warning. This scenario is both undesirable and unacceptable and, as noted previously, may result in damage to adjacent buildings or persons. However, it is difficult to see how inwards collapse - particularly if the panel movement is controlled by the deforming frame - can have an adverse influence on life safety. In order to obtain such deformations, the fire must be well developed within the building - well beyond the stage that firemen would be able to fight the fire from within the building.

Gnanakrishnan (1993) analyses what is purported to be a "standard" wall panel. The panel was a 190 mm thick reinforced concrete wall panel, 8 m in height and subjected to a standard fire exposure. The wall reinforcing details are not given.

The wall was analysed using a special purpose finite element program. The boundary conditions assumed for the wall are illustrated in Figure 2.11. The forces at the connection, (assumed to be at the rafter level) were determined. The lateral supporting structure was considered to be infinitely rigid throughout the fire and the base of the wall rotationally fixed. This is clearly not the case with a supporting roof or frame.

No account was taken of the interaction of the wall curvature with columns - there is a basic incompatibility between the straight column flange and the wall curvature.

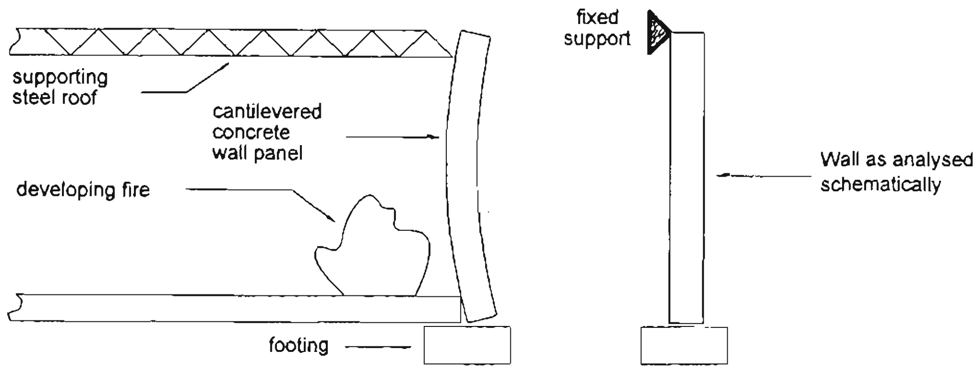


Figure 2.11 Behaviour of Cantilevered Concrete Wall in Tilt-Up Construction; (Gnanakrishnan, 1993)

It is Gnanakrishnan's view that cantilevering the base of the concrete wall is beneficial and will minimise the likelihood of outward collapse. He suggests that in cases where the concrete wall panel is pinned at its base, the connection at the top of the wall must remain intact in order for the wall panel to stay attached to the supporting structure.

2.3.4 Support of Walls

This matter has been considered by both O'Meagher et al. (1990) and Gnanakrishnan (1993).

O'Meagher et al., (1990), consider that integrity of the connection between panels and frame is essential in minimising the likelihood of outwards collapse.

If vertical wall panels are effectively fixed at the base, they will deflect outwards away from the fire, and as noted previously, the steel frames may undergo substantial deformation with one column moving inwards - ie. in the opposite direction to the wall. It is clear, therefore, that the connections must also possess sufficient displacement capacity to allow for such relative displacement. Recent work by O'Meagher et al. (1994), has attempted to establish appropriate design criteria for such connections.

The importance of both strength and ductility to a connections' behavioural aspects is reinforced by some real fire situations ~~illustrated in~~ Appendix A. These issues of strength and ductility are considered in greater detail in Chapters 4 and 5.

Gnanakrishnan (1993), recognises the importance of connection strength but makes no reference to ductility requirements - *"failure of the connections is the most important contributory factor to the instability of the wall panels"*.

It is recognised that the frame columns may deform in the opposite direction to the wall panels, with the possibility of significant forces being developed. Gnanakrishnan found that initially the forces are greatest in the early stages of fire growth and then reduce in magnitude as the structural components become affected by fire. The force in the rigid connection, associated with the situation shown in Figure 2.11, was estimated by Gnanakrishnan to reduce to 50 kN/m after 90 minutes of standard fire exposure. Some serious questions have to be asked about the validity of this result as it seems unlikely that the wall could support a base bending moment of 50 x 8 = 400 kN-m per metre width of wall. Typical walls have an ambient base capacity per metre of approximately 5 - 10 kN-m per metre width of wall. As noted above, the reinforcing details associated with the analysed wall are unknown. It appears unlikely therefore, that the above result can be considered as representative of normal construction.

Moreover, the restraint force will be limited by the ability of the supporting member to plastically deform and the degree of deformation that the frame experiences. These factors were ignored by Gnanakrishnan.

Due to the high temperatures that may be achieved by the supporting structure, Gnanakrishnan considers it important to keep connections and lateral supporting members cool - and for this reason he recommends that connections and supporting members are provided along the outside face of the wall. It is recommended that a lightweight eaves member can be run around the outside of the building as illustrated in Figure 2.12. This member is to be designed to resist the lateral forces applied by the outwardly bowing concrete wall panels.

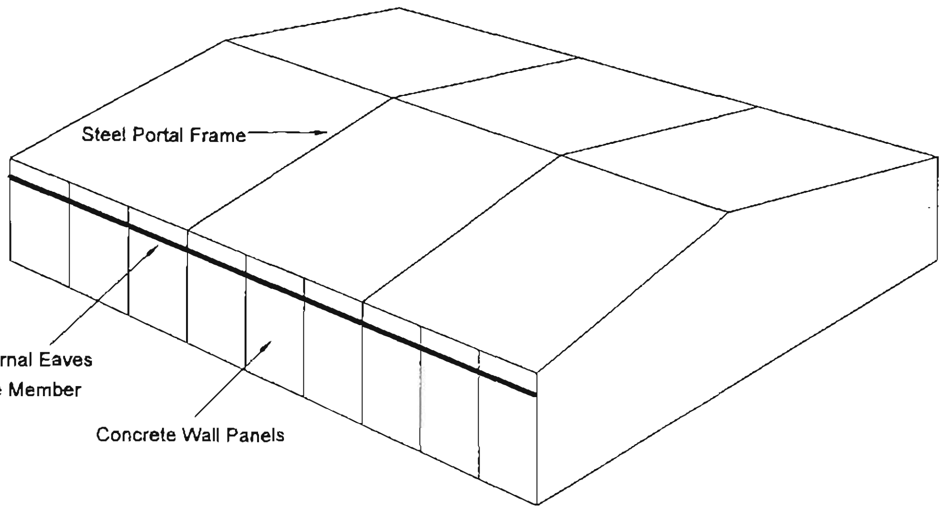


Figure 2.12 External Eaves Tie Member

However, the authors considers this recommendation to have limited appeal due to the additional cost, architectural appearance, and concerns about corrosion. Furthermore, this system will only work provided the eaves member is capable of acting as a catenary member - a light channel may not have sufficient bending strength. The member should be connected to each frame and at the end of the building would need to be properly anchored. These aspects are not mentioned by Gnanakrishnan who appears to assume that the eaves member can span from one end of the building to the other. This could only be achieved if the member acts as a cable and would require effective anchorage at the ends of the building (see Figure 2.13). It is extremely difficult to see how the eaves member or anchorages could resist a lateral force of 50 kN/m.

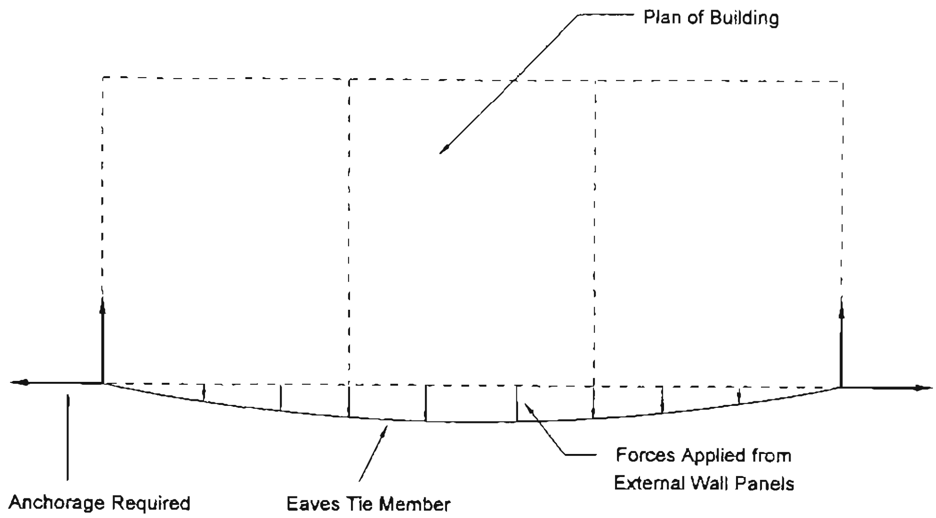


Figure 2.13 Anchorage of External Eaves Tie Member

Alternatively, O'Meagher et al. (1990) consider that the use of non-fire rated connections is not a major problem with regards to providing adequate structural behaviour provided they have sufficient strength and ductility.

Gnanakrishnan also considers the use of nylon bolts to connect the wall panels to the supporting structure. If the panels are cantilevered, the use of such bolts, it is argued, will allow the separation of structure and panels such that the panels will continue to resist the fire effects through cantilever action. The unprotected steel frame will therefore, in theory, be allowed to collapse without affecting the structural adequacy of the external walls.

In the opinion of the author this is a dubious approach - especially for practical walls with minimum quantities of reinforcement. As will be shown in Chapter 4, these walls do not exhibit a high level of fire-resistance as isolated cantilevers, and given a fire of sufficient magnitude, may well collapse outwards.

2.4 Summary of Design Philosophies

The different design philosophies are now summarised as follows:

According to the Constrado publication, (1980):

1. portal frame rafters do not require fire protection;
2. the foundations, column and column base should be designed to resist the overturning moment at the base of the column - assuming a nominated frame geometry;
3. protection of the column has always been required in the United Kingdom. The validity of this requirement was not addressed.

Alternatively, Gnanakrishnan (1993) gives the following design recommendations that:

1. adequate base capacity for the wall must be provided;
2. the provision of a continuous eaves tie should be provided on the outside of the building - no details are given regarding connection of this external tie member to the supporting columns;
3. the size of the above mentioned eaves tie should be designed to resist a force of approximately 50 kN/m per metre width of wall. As noted previously, it is difficult to see how forces of this magnitude can be achieved or resisted.

On the other hand, O'Meagher et al. (1994) have suggested the following philosophy:

1. the steel frames, in the event of a fire, will deform in a satisfactory manner;
2. provided the panels remain attached to the portal frame they will not collapse outwards;
3. the connections between the steel supporting structure an the concrete panels must be designed to have sufficient ductility and adequate strength at elevated temperature;
4. adequate allowance for relative movement between the concrete wall panel and the structural steel portal frame must be made to allow for the difference in wall and frame deformations.

CHAPTER 3 MODELLING OF CONCRETE WALLS IN FIRE

3.1 Introduction

In this chapter a mathematical model is developed to study the structural behaviour of reinforced concrete walls when subject to elevated temperatures due to fire on the internal face.

As highlighted in the previous chapters, the interaction between the supported external walls and the supporting steel structure is of critical importance in determining whether the separation of wall and structure will occur, and it is the deformation and resistance of the external walls that, to a large extent, determines the forces developed at the connections between the frame and the wall. The analysis model described in this chapter is used in Chapter 4 to study the behaviour of some typical walls and to provide some insights into the design requirements for connections.

In developing the mathematical model described in this chapter, a number of convergence problems were encountered. These problems are discussed.

3.2 Elevated Temperature Analysis

To model the behaviour of concrete walls subject to elevated temperatures, it is necessary to determine the temperature distribution across the wall at any given time. This is achieved through analysing the heat flow characteristics through the concrete wall for a given imposed heating situation.

The model developed in this chapter assumes that the wall is subjected to heating on one side only. This is appropriate for the situation being considered as the fire is either inside or located external to the building. The former situation will generally subject the wall and the structure to the most intense heat and is therefore exclusively considered in this study.

The calculation of temperatures throughout the wall was undertaken using the finite element program TASEF-2 (Wickström, 1983). The program is a transient two-dimensional heat flow analysis and enables a number of materials to be considered, including concrete and steel. Conservative thermal properties (i.e. properties likely to give higher than normal temperatures for thermal conductivity, heat capacity and moisture content) are adopted for concrete. Any heating situation can be imposed on one face of the wall by specifying a time-temperature relationship, and appropriate values of radiative and convective heat transfer coefficients. Specific coefficients are recommended for simulating the standard fire test heating environment (Wickström, 1983).

In practice, as noted in the previous chapter, real fires are highly variable. For the purpose of the analysis conducted in Chapter 4, the fire is assumed to correspond to an ISO standard fire of some duration. The use of the ISO standard fire exposure relationship is adopted not only for simplicity but also because it will enable a good estimate of the relative performance of the concrete walls subject to various fire scenarios and because it is the basis for current regulatory requirements for walls. The vast majority of fire engineering designs are based at present on the standard fire exposure (Twilt, 1988).

In the standard fire test it is required that the temperature-time relationship follows the following equation.

$$T_f = T_o + 345 \log_{10} (480t + 1) \qquad \text{Eq. (3.1)}$$

where:

T_f	:	Fire Temperature (°C)
T_o	:	Ambient Temperature (°C)
t	:	Duration of Fire Exposure (min)

To calculate the temperatures through the wall using TASEF-2, the wall must be discretised across its thickness into a number of elements. The appropriate fire-time temperature curve and heat transfer coefficients, and the material thermal properties must also be specified. Comparison of temperatures predicted using TASEF-2 with measured temperatures obtained during standard fire tests (The Institution of Structural Engineers, 1978) indicate good agreement, with the predicted temperatures being generally greater nearer to the heated surface; see Figure 3.1 .

The temperatures through a 150 mm wall, when subject to 30, 60 and 90 minutes of standard fire test exposure on one side of the wall are shown in Figure 3.1.

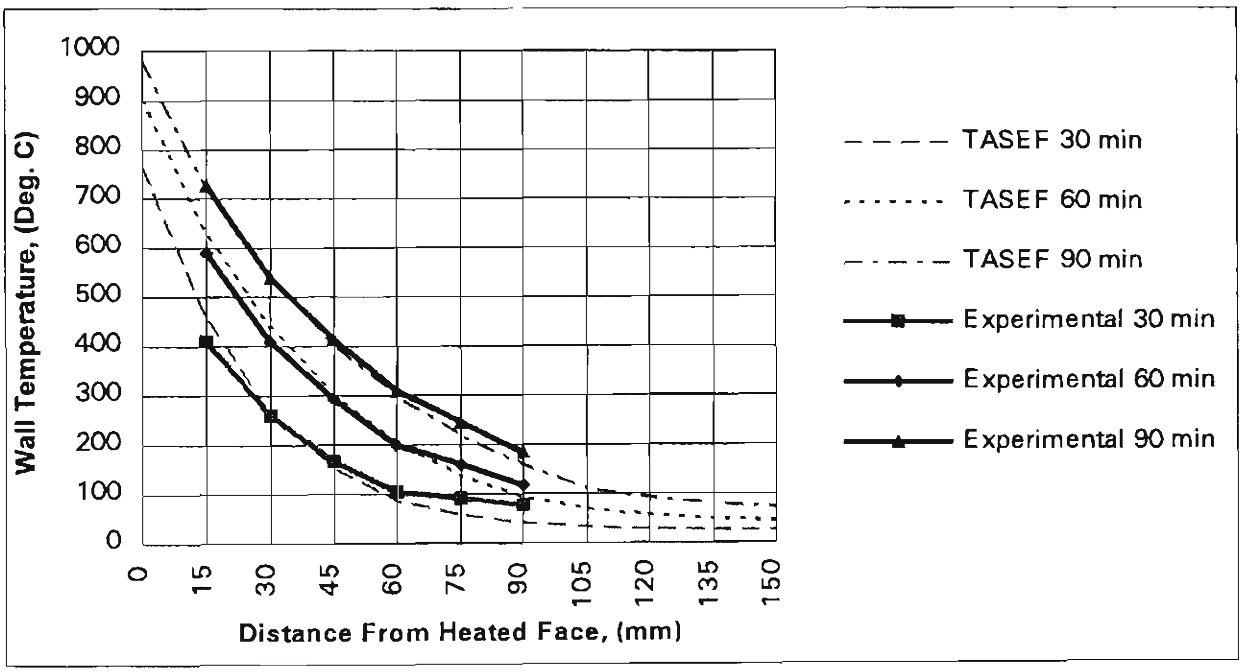


Figure 3.1 Standard Fire Temperature Gradients at Varying Times of Exposure for 150 mm Thick Concrete Wall - [TASEF-2 versus Experimental Results]

At the interface, between the wall and the fire, the temperature is assumed to be that associated with the fire. The temperatures away from this junction fall rapidly. The temperature gradient varies with time of fire exposure, being greatest for 30 minutes. This suggests that most of the thermally induced wall deformations will occur during the earlier stages of fire exposure.

3.3 Modelling of the Wall

For the purpose of developing a mathematical model, the cases shown in Figure 3.2 have been considered. The situation shown in (a) represents a vertical wall panel which is effectively rotationally restrained at the base and subject to a horizontal load at the top of the wall in addition to its own self weight acting in a vertical direction (see also Figure 3.3). As will be demonstrated in Chapter 4, this situation always occurs where a vertical panel is connected to a steel frame such that it is directly adjacent to a column. The situation shown in (b) represents a horizontal wall panel which is partially rotationally restrained at each end of the panel at the column support via a clip connection. This often occurs where a horizontal panel is connected to a steel frame.

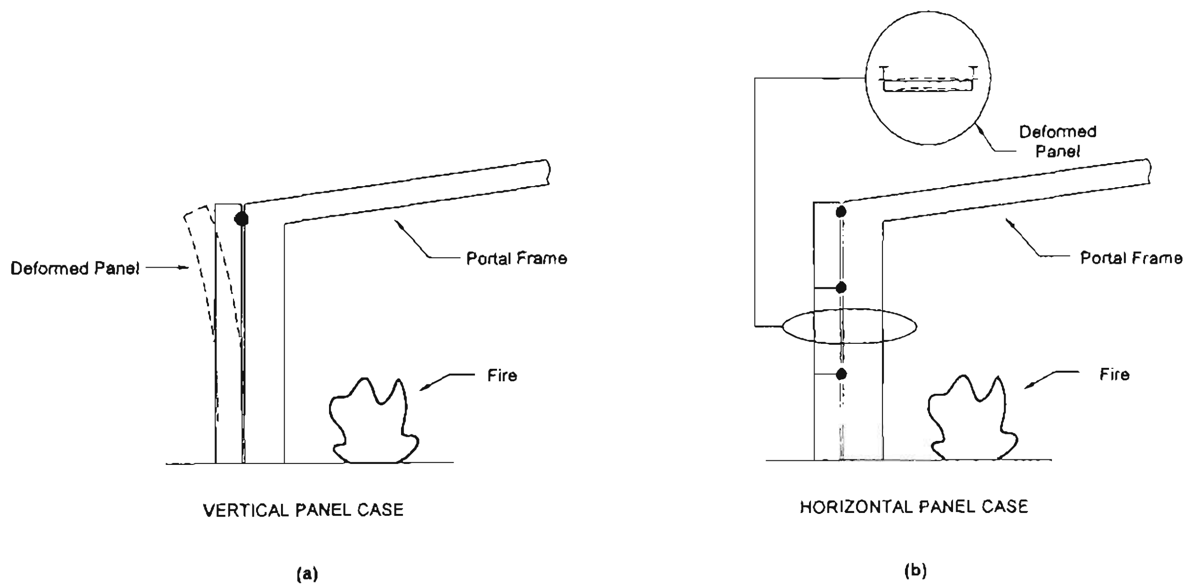


Figure 3.2 The Two Cases Analysed for the Wall Model Study

The above cases were modelled as shown in Figure 3.3 .

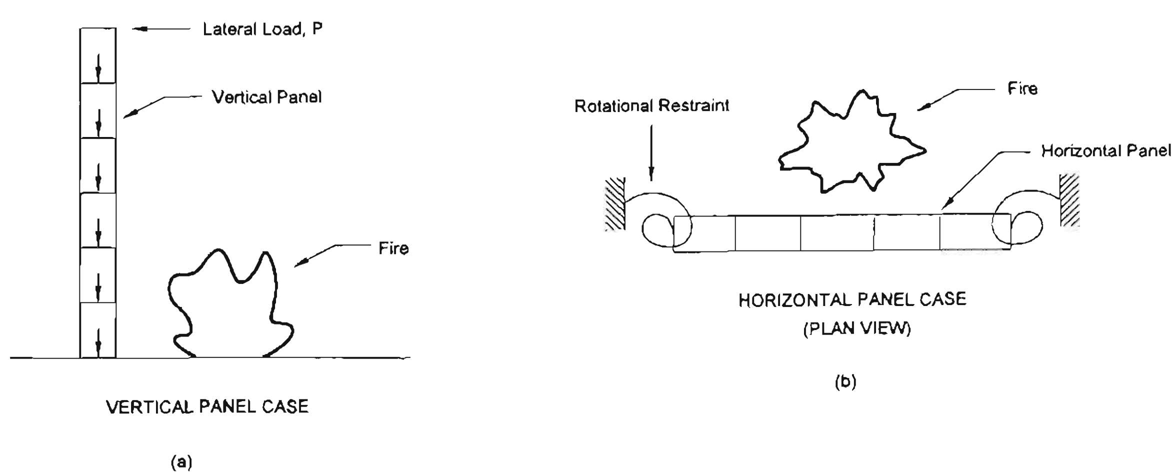


Figure 3.3 The Two Cases as Modelled for the Analysis

The method of analysis presented in this chapter assumes that the wall can be analysed as a one-way member with curvature in only one direction. This is likely to lead to an over-estimate of displacements. Nevertheless, it is a reasonable and conservative assumption.

3.3.1 Discretisation of Wall Model

For the purpose of analysis, the wall is discretised across its thickness and length as shown in Figure 3.4.

The analysis program allows for different numbers of elements and segments.

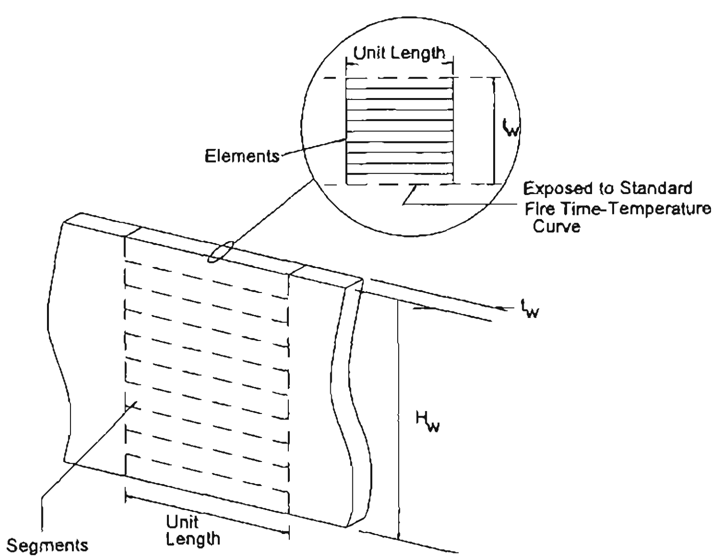


Figure 3.4 Discretisation of the Cantilevered Concrete Wall

The effect of increasing the number of segments up the wall has been investigated and the results of this analysis are given in Appendix D.

3.3.2 Constitutive Equations

In order to determine the structural behaviour of the concrete wall under elevated temperature conditions, it is necessary to allow for the effect of elevated temperature on the mechanical properties of both the steel reinforcing and the concrete.

According to Anderberg (1973), the strain state associated with a concrete element can be broken up into the following:

$$\varepsilon_{tot} = \varepsilon_{th} + \varepsilon_{\sigma} + \varepsilon_{cr} + \varepsilon_{tr} \qquad \text{Eq.(3.2)}$$

where:

ε_{tot}	=	total strain of the specified element
ε_{th}	=	free thermal expansion of the material at a given temperature
ε_{σ}	=	stress related strain
ε_{cr}	=	creep, (or time-dependent), strain of the concrete and/or steel
ε_{tr}	=	transient strain of the material

In equation 3.2 above, ε_{th} is a function of temperature, ε_{σ} is a function of stress level and temperature, ε_{cr} is a function of stress level, time, and temperature, and ε_{tr} considers the effect of stress history on deformation at elevated temperatures.

The strain state within the steel reinforcing element can be represented in a similar manner - although in this case, transient strain is not appropriate.

For the purposes of the model developed in this chapter, the thermal expansion relationships recommended in Eurocode 4 (Schleich et al., 1990) have been adopted whilst the stress-strain relationships recommended in Eurocode 2 (Dotreppe et al., 1990) have been adopted. Typical stress-strain relationships for reinforcing steel and concrete are shown in Figures 3.5 and 3.6, respectively.

For concrete, the transient strain term, ε_{tr}, and the creep strain term, ε_{cr}, have been implicitly (and appropriately) included in the Eurocode 4 stress-strain relationships. Thus, only thermal and stress related strains are explicitly considered.

For steel reinforcement, creep effects are only significant for temperatures greater than 450 °C (Williams-Leir, 1983).

Again, the creep effects are implicitly included in the Eurocode 4 stress-strain curves for steel. It is important to note that Eurocode 4 recommends different stress-strain relationships for reinforcing steels - depending on whether the steel is cold-worked or not. Reinforcing fabric is manufactured from hard drawn wire and is most commonly used to reinforce concrete walls. It is rare that Y bars are used.

Therefore the appropriate stress-strain relationships for steel reinforcement, for the analyses undertaken in Chapter 4, are those corresponding to cold-worked reinforcing steel as shown in Figure 3.5.

It has been assumed that use of the above European stress-strain relationships will give an adequate representation of the elevated temperature behaviour of Australian materials.

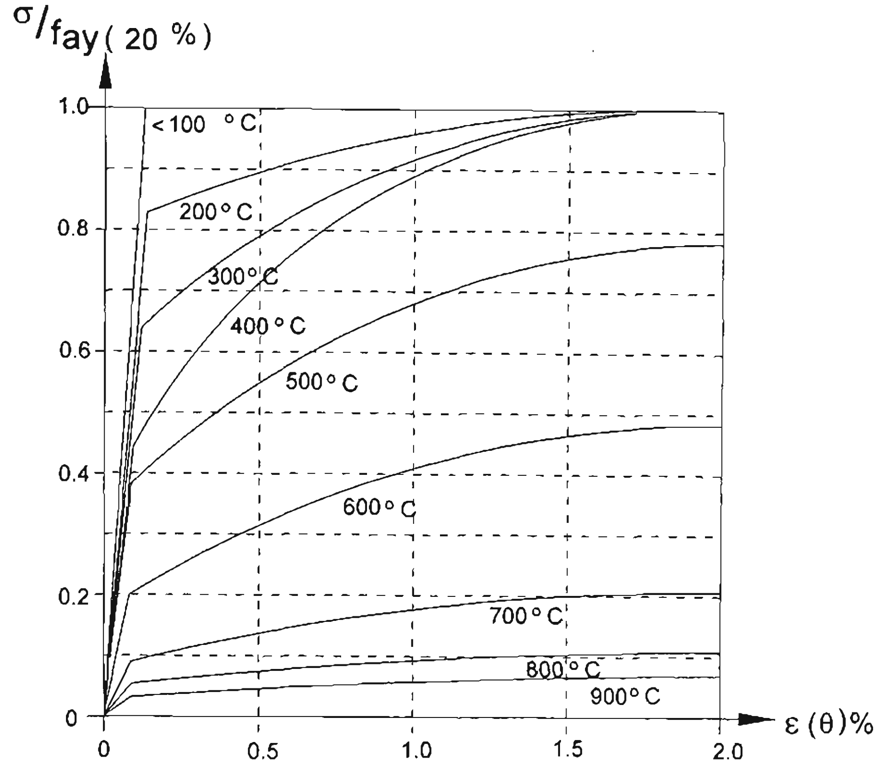


Figure 3.5 Stress-Strain Relationships for Reinforcing Steels at Elevated Temperatures

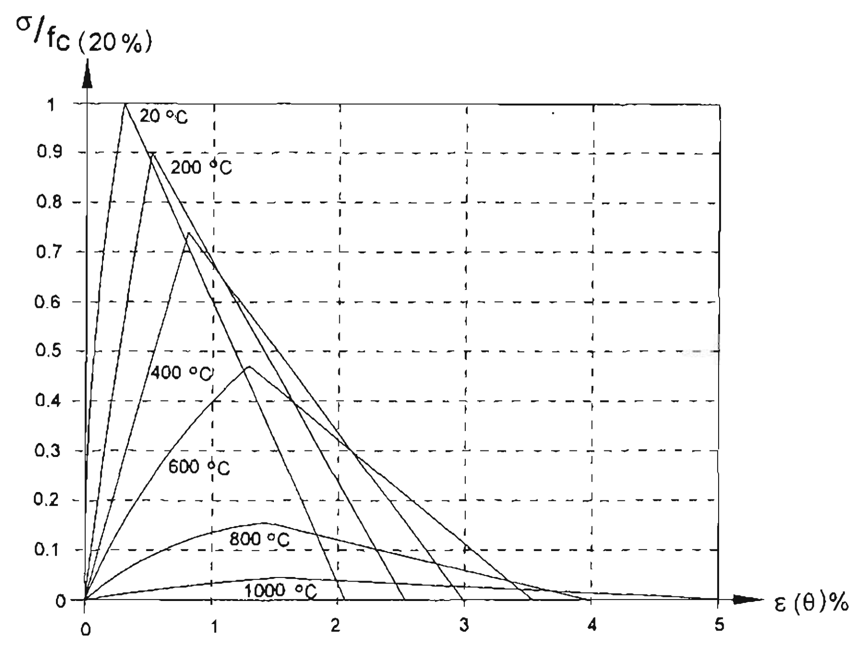


Figure 3.6 Stress-Strain Relationships for Siliceous Concrete at Elevated Temperatures

The majority of concrete used in Australia in the manufacture of concrete walls use siliceous aggregates. The relationships given in Figure 3.6 are therefore appropriate.

3.3.3 Strain State within A Segment

Within any segment down the wall it is assumed that the deformation of the elements is such that plane sections before deformation remain plane after deformation.

It is also assumed that the curvature within a segment is constant over the length of the segment.

With the strain state within a segment assumed to be constant, the stress state can be determined through satisfaction of the equilibrium equations given below in Section 3.3.4.

3.3.4 Equilibrium Equations

The equilibrium situation associated with a deformed wall is shown in Figure 3.7 and is described by the following governing equations.

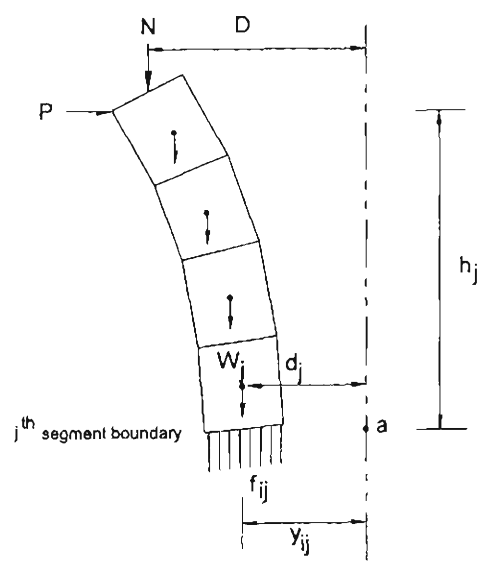


Figure 3.7 Equilibrium Equations for the Cantilevered Wall

where:

'a' = point adjacent to segment boundary about which moments are taken

i = element number

j	=	segment / boundary number
n_i	=	total number of elements analysed
n_j	=	total number of segments analysed

At each segment boundary the following equilibrium equations must be satisfied:

The Resultant Force, Fr, at the j^{th} segment boundary = 0 :

i.e. $F_r = 0$

But, $F_r = F_{int} - F_{ext}$ Eqs. (3.3)

Equating the above equations, we have:

$$F_r = \left(\sum_{i=1}^{n_i} f_{ij}\right) - \left(\sum_{j=1}^{n_j} W_j + N\right) = 0$$

Eq. (3.4)

The Resultant Moment, Mr, at the j^{th} segment boundary about 'a' = 0

i.e. $M_r = 0$

But, $M_r = M_i - M_e$ Eqs. (3.5)

Equating the above equations, we have:

$$M_r = \left(\sum_{i=1}^{n_i} f_{ij} \cdot y_{ij}\right) - \left(\sum_{j=1}^{n_j} W_j \cdot d_j - P \cdot h_j + N \cdot D\right) = 0$$

Eq. (3.6)

For the wall to be in equilibrium each segment boundary must satisfy the above equations.

For the model described above, equilibrium was progressively checked at the various element boundaries starting from the top of the wall and working downwards, (see Figure 3.7).

3.3.5 Compatibility Equations

In addition to the equilibrium equations, it was necessary to determine the relationships between segment rotation, curvature and lateral displacement.

Given the curvatures of the wall at each of the segment boundaries, the deflected shape can be determined using the following equations; see Figure 3.8.

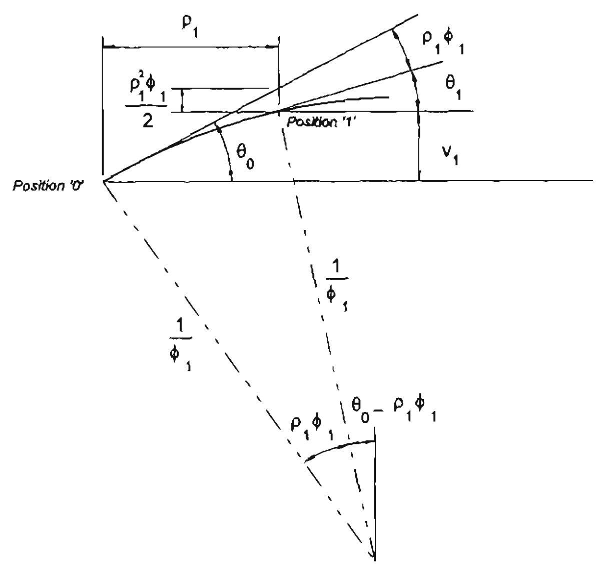


Figure 3.8 Geometric Relationships Between Successive Points on a Curve

where:

θ	is the slope at the element boundary
ϕ	is the curvature at the element boundary
ρ	is the element length
v	is the lateral displacement at the element boundary

The compatibility equations are expressed as follows:

$$v_1 = \rho_1\theta_0 - \frac{\rho_1^2\Phi_1}{2}$$

$$\theta_1 = \theta_0 - \rho_1\Phi_1$$

$$\nu_2 = \nu_1 + \rho_2\theta_1 - \frac{\rho_2^2\Phi_2}{2}$$

$$\theta_2 = \theta_1 - \rho_2\Phi_2 \qquad \text{Eqs. (3.7)}$$

and so forth with the deflection and the rotation of the j^{th} point determined as follows:

$$\nu_j = \nu_{j-1} + \rho_j\theta_{j-1} - \frac{\rho_j^2\Phi_j}{2}$$

$$\theta_j = \theta_{j-1} - \rho_j\Phi_j \qquad \text{Eq. (3.8)}$$

For a cantilevered wall, the displacement and rotation at the base of the wall are zero. The displacements and rotations for points above the base are easily determined from the above geometric equations.

3.4 Method of Solution

The non-linear behaviour of concrete and steel at elevated temperatures, and the fact that Equations 3.3 - 3.8 are interdependent, means that an iterative solution procedure must be adopted.

Newton's Iteration Method, (La Fara, 1973), was used for solving the above simultaneous equations. This method is described below.

Assuming the resultant axial force (refer to Equation 3.4) associated with equilibrium of a wall cross section is represented by the symbol F_r, and the resultant bending moment about 'a' (see Figure 3.7) resulting from the internal and external forces is given by M_r (refer to Equation 3.6), the following general equilibrium expressions can be written:

$$F_r(\varepsilon,\phi)=0$$
$$M_r(\varepsilon,\phi)=0 \qquad \text{Eq. (3.9)}$$

where:

ε is the external surface strain of the segment j ; (see Figure 3.9 below)

ϕ is the curvature of the segment j ; (see Figure 3.9 below)

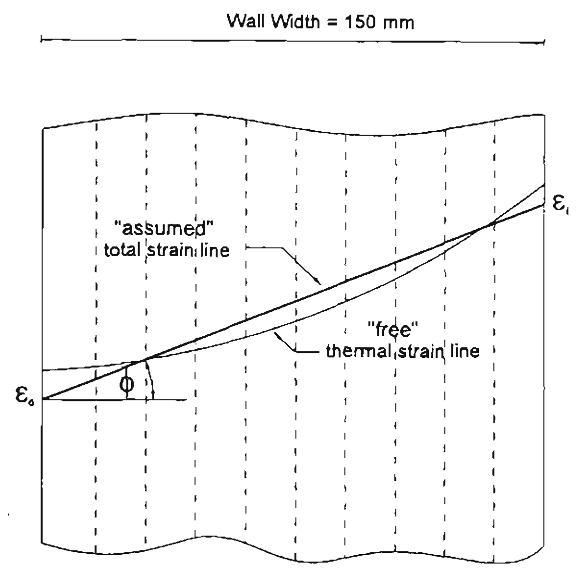


Figure 3.9 Locations of Initial Strain and Curvature Estimates at Segment j.

If ε and ϕ are the solutions, and ε^1 and ϕ^1 are some initial estimates, then;

$$\varepsilon=\varepsilon^{(1)}+\Delta\varepsilon$$
$$\phi=\phi^{(1)}+\Delta\phi \qquad \text{Eq. (3.10)}$$

Substituting equations 3.9 into equations 3.10,

$$F_r\left(\varepsilon^{(1)}+\Delta\varepsilon\ ,\ \phi^{(1)}+\Delta\phi\right)=0$$
$$M_r\left(\varepsilon^{(1)}+\Delta\varepsilon\ ,\ \phi^{(1)}+\Delta\phi\right)=0 \qquad \text{Eqs. (3.11)}$$

Using Taylor's series and ignoring terms that are of order greater than one, the following approximate relations are obtained:

$$F_r\left(\varepsilon^{(1)},\phi^{(1)}\right)+\Delta\varepsilon\left(\frac{\partial F_r}{\partial\varepsilon}\right)^{(1)}+\Delta\phi\left(\frac{\partial F_r}{\partial\phi}\right)^{(1)}=0$$

$$M_r\left(\varepsilon^{(1)},\phi^{(1)}\right)+\Delta\varepsilon\left(\frac{\partial M_r}{\partial\varepsilon}\right)^{(1)}+\Delta\phi\left(\frac{\partial M_r}{\partial\phi}\right)^{(1)}=0$$

Eqs. (3.12)

In order to obtain the above partial derivatives, three trials of initial strain (ε), and curvature (ϕ), were undertaken to commence the iterative process. That is, three trials were undertaken to obtain a first estimate of partial derivatives (see Equation 3.14).

Trial 1	$F_{i_1}\left(\varepsilon_o\ ,\ \phi_o\right),\ M_{i_1}\left(\varepsilon_o\ ,\ \phi_o\right)$
Trial 2	$F_{i_2}\left(\varepsilon_o\ ,\ \phi_o+\Delta\phi\right),\ M_{i_2}\left(\varepsilon_o\ ,\ \phi_o+\Delta\phi\right)$
Trial 3	$F_{i_3}\left(\varepsilon_o+\Delta\varepsilon\ ,\ \phi_o\right),\ M_{i_3}\left(\varepsilon_o+\Delta\varepsilon\ ,\ \phi_o\right)$

Eqs. (3.14)

These first trials were then used to obtain partial derivatives as follows:

$$\frac{\delta F_r}{\delta\varepsilon}=\frac{F_{i_3}-F_{i_1}}{\Delta\varepsilon}\quad,\quad\frac{\delta F_r}{\delta\phi}=\frac{F_{i_2}-F_{i_1}}{\Delta\phi}$$

$$\frac{\delta M_r}{\delta\varepsilon}=\frac{M_{i_3}-M_{i_1}}{\Delta\varepsilon}\quad,\quad\frac{\delta M_r}{\delta\phi}=\frac{M_{i_2}-M_{i_1}}{\Delta\phi}$$

Eq. (3.15)

$$\text{Det}=\frac{\delta F_r}{\delta\varepsilon}\cdot\frac{\delta M_r}{\delta\phi}-\frac{\delta F_r}{\delta\phi}\cdot\frac{\delta M_r}{\delta\varepsilon}$$

Eq. (3.16)

Values of $\Delta\varepsilon$ and $\Delta\phi$ were obtained using Eq. (3.12) which when written in matrix form gives:

$$\begin{bmatrix} \dfrac{\partial F_r}{\partial \varepsilon} & \dfrac{\partial F_r}{\partial \phi} \\ \dfrac{\partial M_r}{\partial \varepsilon} & \dfrac{\partial M_r}{\partial \phi} \end{bmatrix} \cdot \begin{bmatrix} \Delta\varepsilon \\ \Delta\phi \end{bmatrix} = \begin{bmatrix} -F_r \\ -M_r \end{bmatrix}$$ Eq. (3.17)

Substituting these estimates of $\Delta\varepsilon$ and $\Delta\phi$ into Eq. (3.10) and inverting the above equation gives:

$$\text{New } \varepsilon_{new} = \varepsilon_{prev} + \left(\frac{-\dfrac{\partial M_r}{\partial \phi} \cdot F_{i_1} + \dfrac{\partial F_r}{\partial \phi} \cdot M_{i_1}}{Det} \right)$$ Eq. (3.18)

$$\text{New } \phi_{new} = \phi_{prev} + \left(\frac{\dfrac{\partial M_r}{\partial \varepsilon} \cdot F_{i_1} - \dfrac{\partial F_r}{\partial \varepsilon} \cdot M_{i_1}}{Det} \right)$$ Eq. (3.19)

The entire process is repeated for each successive cycle of iteration (i.e., $i \geq 2$). The iterative process is continued until negligible change occurs between successive cycles for calculated displacements.

3.4.1 Analysis Procedure

The solution procedure is illustrated in Figure 3.7 and the computer coding is given in Appendix C.

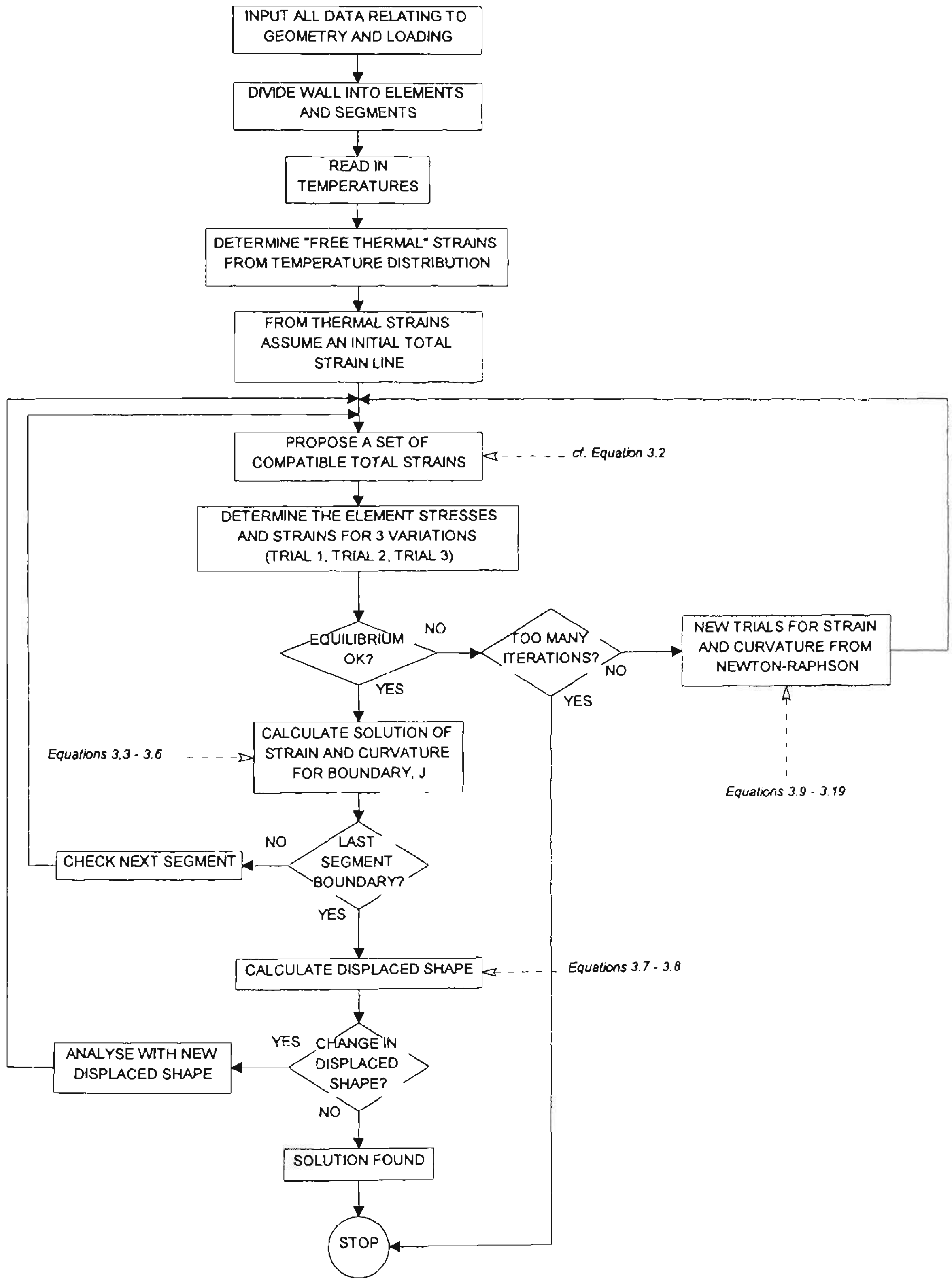


Figure 3.10 Analysis Flow Chart for the Solution of the Wall Model

3.4.2 Problems Associated with the Iterative Procedure

Some difficulties were encountered with the above solution procedure. Newton's iteration procedure can be sensitive to the starting point. This was found to be the case in some situations where the choice of starting curvature of strain sometimes caused increasing divergence when a solution actually existed. Careful choice of starting values was required.

3.5 Comments on Model

The wall model developed in this chapter is similar to that developed by O'Meagher and Bennetts (1991) and is based on similar assumptions.

As noted previously, the model developed in this chapter utilises the Eurocode 4 elevated temperature properties for both steel and concrete. These are, in principle, much simpler than the complex models incorporated in O'Meagher and Bennetts' model. The Eurocode 4 models are generally recognised as giving larger displacements and less fire-resistance than that which would be achieved using more sophisticated material models.

A comparison of a situation analysed by O'Meagher and Bennetts, (1991) and that analysed using the model in this thesis is given below.

Concrete Wall Analysed	Wall Thickness	150 mm
	Wall Height	6.5 m
	Wall Length	7.5 m
	Concrete Strength	25 MPa
	Steel Reinforcement	450 mm^2/m (centrally located)
	Steel Yield Strength	400 MPa
	Axial Load	15.6 kN

	O'Meagher and Bennetts (1991)	Bortoli (1995)
At 30 minutes SFT exposure:	$\Delta \approx 975$ mm	$\Delta \approx 1076$ mm

It can be seen that with the wall being uniformly heated over it's entire length, the values of displacement at the top of the wall compare quite favourably; i.e. <10% difference.

3.6 Conclusion

A numerical model for analysing the large displacement behaviour of concrete walls in fire has been developed. The model utilises the "simplified" Eurocode 4 stress-strain relationships for both concrete and steel.

It is found to give conservative (i.e. larger) displacements than those associated with the model developed by O'Meagher and Bennetts (1991).

CHAPTER 4 STUDY OF WALL DEFORMATIONS

4.1 Introduction

The purpose of this chapter is to further study the relationship between wall deformations and fire severity; ie. how far does the wall move laterally for a given fire exposure.

Through utilising the wall model developed in Chapter 3, a greater understanding of the magnitude of deformations relating to various fire exposures is gained. Also the interaction between the lateral restraining systems, including connections, and the deforming wall can be further understood.

The chapter examines the deformation behaviour of both unloaded and loaded concrete walls under varying heat exposures and varying heights of exposure. The behaviour of both horizontal and vertical panels is considered.

4.2 Vertical Panels

4.2.1 Unrestrained Wall Deformation - Self Weight Only

4.2.1.1 Introductory Concepts

In order to determine the structural response of a vertical wall element to fire, it is necessary to study the behaviour of an unrestrained wall. In this case, the effects of thermal gradient and wall self weight only, are taken into account with the walls analysed as cantilevered members.

Two levels of reinforcement have been considered for the analysis; the level specified in Section 11 - AS 3600 (the typical case) and twice this level. Hereafter, these walls are referred to as "typical panels".

The relevant details for the wall panels considered are:

Wall Height	6 m and 8 m
Wall Width	1000 mm
Wall Thickness	150 mm
Concrete Grade, f'_c	32 MPa
Reinforcing Steel	1) Minimum Reinforcement; Reinforcement Ratio, p_w = 0.0015 % ≡225 mm^2/m *(as per AS 3600 Section 11).* 2) Double Reinforcement; Reinforcement Ratio, p_w = 0.0030 % ≡450 mm^2/m
Steel Yield Stress, f_{sy}	450 MPa
End Conditions	Free at the top, Fixed at the bottom
External Loads	Lateral Load - optional Axial Load - optional
Load Eccentricity	Axial load is assumed to act centrally through the wall

Using the model developed in Chapter 3, the walls were subjected to varying levels of standard fire exposure, with periods of exposure ranging from 2.5 to 90 minutes duration.

As noted previously (see Chapter 2), the characteristics of fires in warehouse buildings are highly variable and, generally, will not resemble standard fire exposure conditions. The real fire time-temperature curve and resultant wall exposure level is likely to be significantly different to those associated with the standard fire test (SFT). Nevertheless, the gas time-temperature curve associated with the standard fire test is conveniently adopted to enable insight into aspects of wall behaviour and evaluation of the relative importance of various factors.

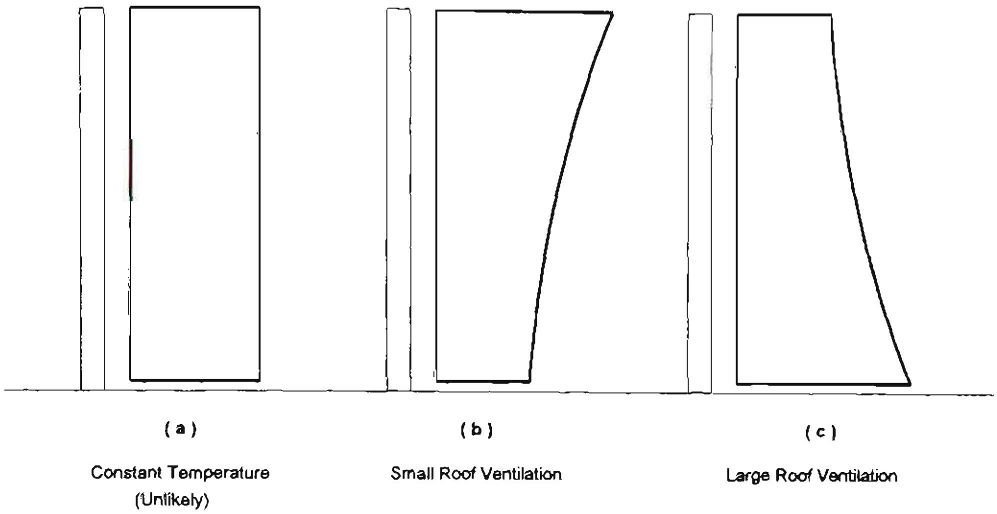


Figure 4.1 Temperature Distributions of Cases Considered in Modelling

In a real fire, it is unlikely that a vertical panel will be exposed to uniform gas temperatures over its height (see Figure 4.1(a)), although this case forms the basis of the work presented in this chapter. Substantial gradients would be expected (Figure 4.1(b) and (c)), and to get some insight into the influence of such gradients up the wall, two extreme partial fire exposure conditions have been considered.

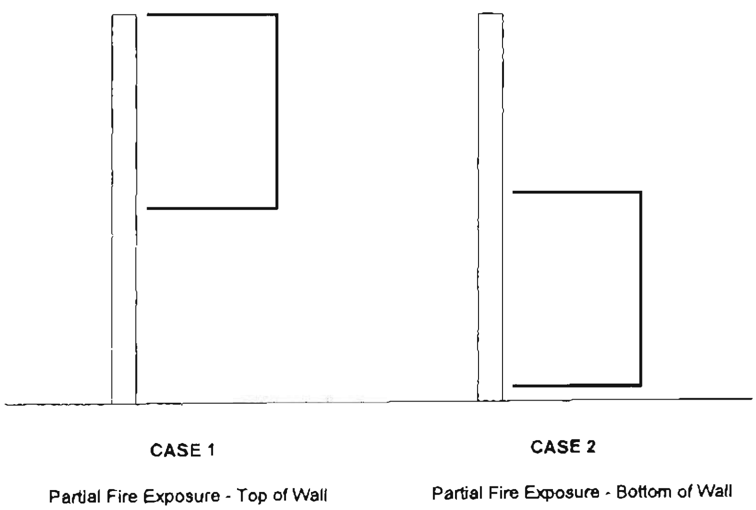


Figure 4.2 Simplified Cases Analysed for Differential Heating

In the first case (Figure 4.2(a)), the wall is assumed to be subjected to the ISO fire over its top half, and in the second (Figure 4.2(b)), the wall is assumed to be subjected to the ISO fire over its bottom half. In both of the above cases, the unheated portions of wall are assumed to remain at ambient temperature.

4.2.1.2 Analysis Results and Discussion

The results of the above analyses are given in Tables 4.1 - 4.6 below.

***Note:** Shaded areas within the below tables indicate wall failure at these exposure conditions.*

Table 4.1 Displacements for 6m Wall, Whole Wall Exposure; (mm)

Reinforcement Ratio; Pw	**Duration of ISO Fire Exposure; (min)**							
	5	***10***	***15***	***20***	***25***	***30***	***60***	***90***
0.0015	194	431	655	782	917	1054		
0.0030	189	406	579	662	727	772	1056	1248

Table 4.2 Displacements for 8m Wall, Whole Wall Exposure; (mm)

Reinforcement Ratio; Pw	**Duration of ISO Fire Exposure; (min)**							
	5	***10***	***15***	***20***	***25***	***30***	***60***	***90***
0.0015	367							
0.0030	349	790	1199	1435	1802			

Table 4.3 Displacements for 6m Wall, Top Half Wall Exposure; (mm)

Reinforcement Ratio; Pw	**Duration of ISO Fire Exposure; (min)**							
	5	***10***	***15***	***20***	***25***	***30***	***60***	***90***
0.0015	51	99	153	187	219	244	284	324
0.0030	48	95	129	141	150	166	224	263

Table 4.4 Displacements for 8m Wall, Top Half Wall Exposure; (mm)

Reinforcement Ratio; Pw	**Duration of ISO Fire Exposure; (min)**							
	5	***10***	***15***	***20***	***25***	***30***	***60***	***90***
0.0015	85	187	285	357	431	489	559	637
0.0030	83	174	243	278	317	336	461	511

Table 4.5 Displacements for 6m Wall, Bottom Half Wall Exposure; (mm)

Reinforcement	Duration of ISO Fire Exposure; (min)							
Ratio; Pw	*5*	*10*	*15*	*20*	*25*	*30*	*60*	*90*
0.0015	135	288	433	538	633	706	827	944
0.0030	132	269	366	403	429	475	650	762

Table 4.6 Displacements for 8m Wall, Bottom Half Wall Exposure; (mm)

Reinforcement	Duration of ISO Fire Exposure; (min)							
Ratio; Pw	*5*	*10*	*15*	*20*	*25*	*30*	*60*	*90*
0.0015	240	538	834	1044	1263	1441	1657	1882
0.0030	234	500	702	812	925	985	1356	1509

Table 4.1 and 4.2 illustrate that the magnitude of displacement at the top of the wall increases with increased level of ISO fire exposure. This displacement can be significant; (eg. 1802 mm after 25 minutes exposure).

Tables 4.1 and 4.2 show that for the 6 m wall, subject to full height ISO fire exposure, the fire-resistance period achieved is 30 - 90 minutes depending on the reinforcement level. In the case of the 8 m wall, the fire-resistance of the free-standing wall is only 5 - 25 minutes. As would be expected, the maximum displacements attained for that of the 8 m high concrete wall panel are larger than those of the 6 m high wall panel for the same fire-resistance level.

The 8 m wall panel has a lesser fire-resistance than the 6 m wall panel. These levels of fire-resistance are well below the level of 90 minutes required for Type C construction; (see Chapter 2).[1]

Of greater significance is the fact that these walls will collapse outwards, (ie. away from the building), if they are designed to act as free-standing members. This is exactly what is intended to be avoided by Clause C1.11 of the BCA.

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1 The wall panels, because of their thickness, will achieve the required levels of fire-resistance with respect to insulation and integrity. The above comment refers only to the matter of structural adequacy.

Tables 4.3 and 4.4 show the effect of varying the ISO fire exposure periods on the maximum lateral displacements, assuming the walls to be subjected to heating of the top half of the wall, whilst Tables 4.5 and 4.6 show this effect when the wall is subjected to heating of the bottom half.

As can be seen from the results given in Tables 4.3 and 4.4, the wall has no difficulty in obtaining a fire-resistance level of 90 minutes when heated over the top half only. As expected, the magnitude of displacements obtained for the case where the wall panel was heated over its top half was significantly less (typically 70-80% less), than the full height exposure case for corresponding fire exposure times. A comparison of Table 4.1 and 4.5 shows that heating of the bottom half also results in a significant increase in levels of fire-resistance (eg. for the minimum reinforced 6m wall the fire resistance level increased from 30 minutes to more than 90 minutes).

Tables 4.5 and 4.6 show that the wall panels, when subject to bottom half heating only, are able to achieve fire-resistances up to and including 90 minutes - the latter with maximum lateral displacements up to 1882 mm. However, the displacements for a given fire exposure time are greater than those for the corresponding "top half" cases.

Although these analyses represent an approximate approach to determining the influence of vertical variation of fire temperature on wall displacement, they illustrate how sensitive wall behaviour is to such variation. By partially heating the wall, the maximum lateral displacements are reduced whilst the level of fire-resistance is improved for a given fire duration.

The influence of a higher percentage of wall reinforcement on fire-resistance and wall displacement can also be observed. Comparison of the results from Table 4.1 shows that the doubling of the area of reinforcement in the wall results in an increase in fire-resistance from 30 minutes to 90 minutes. However, from Table 4.2 it can be seen that the increase in fire-resistance is not as pronounced for the 8 m case as for the 6 m case. As would be expected, the maximum lateral displacements achieved at the top of the wall prior to failure are less for the more highly reinforced walls. For a given fire-resistance exposure time, doubling the amount of reinforcement reduces the maximum lateral displacements at the top of the wall by between 2.5 - 5%.

The above analysis results illustrate that the presence of a temperature gradient up the wall height will increase the fire-resistance of free-standing walls. Nevertheless, it is clear that free standing walls of 8m height with minimum reinforcement (often the case) will not achieve high levels of fire-resistance. Therefore, in the event of a large fire, it is quite possible that these walls will collapse outwards (see Chapter 2). Therefore it is considered that these walls should be tied back to the supporting frame.

It seems reasonable, therefore, to require vertical panels to be tied back to the supporting frame through adequate connection.

4.2.2 Restrained Wall Deformation - Self Weight and Lateral Load

4.2.2.1 Introductory Concepts

Restraint of a wall will result in lateral forces being applied at the top of the wall at the restraint location.

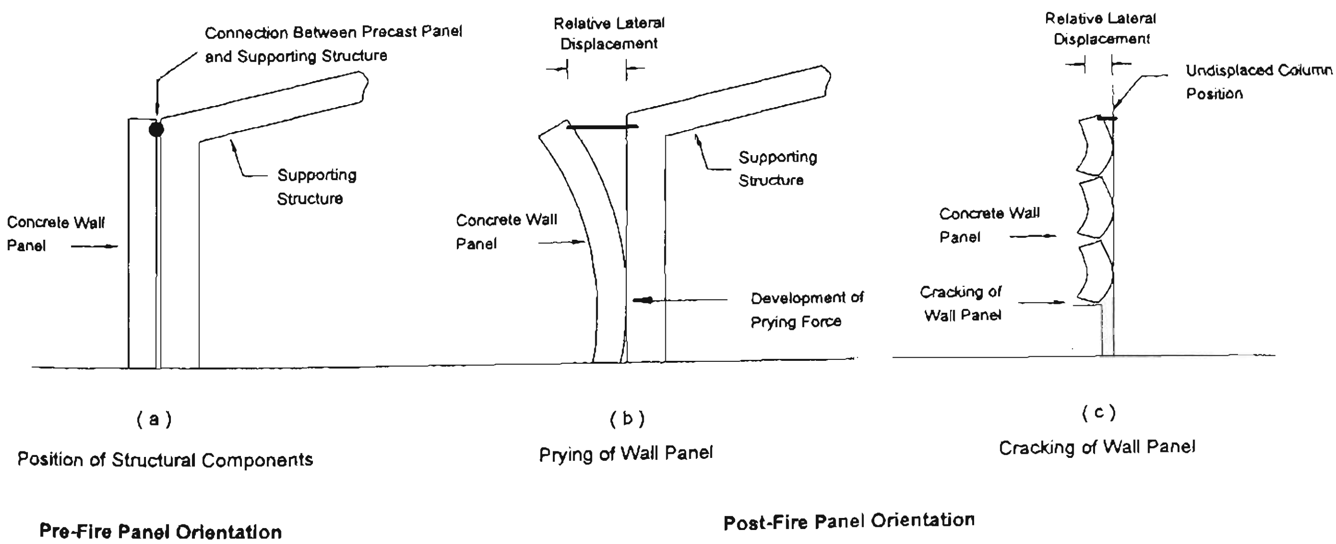


Figure 4.3 Cases Considered for Analysis of Restrained Wall Deformation

Figure 4.3(a) shows an external concrete wall panel tied to a steel frame. What happens when the wall panel is heated? Even if the wall panel is "pinned" at the base it will "pry" against the column flange, see Figure 4.3(b), resulting in the wall attempting to act as a cantilevered wall. As seen from the results of the above analyses, cantilevered walls will attempt to deflect away from the fire. However, if these walls are connected to the frame, then the connection will resist this deformation - and as noted above, a restraining force will be developed.

If the supporting structure and connection is infinitely rigid, then no relative displacement is possible and the wall panel will progressively crack up its' height (see Figure 4.3(c)) and very large forces may be developed at the connection. These forces may be sufficient to fail the connection.

It is necessary, therefore, for the connection between the concrete wall panel and supporting structure to allow some relative displacement without failing. As the fire progresses, this displacement is likely to increase to a maximum value. This value is defined as the limiting relative lateral displacement, δ_{lr}.

But what is an appropriate value of relative displacement for the connections to achieve?

Values of relative limiting displacement at the connection have been proposed by O'Meagher et al. (1994) based on observations of concrete wall panels after actual fires but represent little more than an 'educated' guess. O'Meagher et al. (1994) consider it unlikely that the relative lateral displacement between wall and column will be significantly exceeded prior to the development of large displacements in the supporting frame and venting of the roof.

The following has been suggested by O'Meagher et al. (1990):

For Vertical Panels;

$$\delta_{lr} = 200 \cdot \left(\frac{H_c}{5}\right)^2 \qquad \text{where } H_c \text{ is less than 5 m} \qquad \text{Eq. 4.1}$$

$$\delta_{lr} = 200 \cdot \left(\frac{H_c}{5}\right) \quad \text{where } H_c \text{ is greater than or equal to 5 m} \qquad \text{Eq. 4.2}$$

For 6 m and 8 m high vertical panels, the relative limiting displacements are 240 mm and 320 mm respectively. In practice, some horizontal movement is also provided by expansion of the rafter but this effect has been ignored for the purpose of this discussion.

It is of interest to determine the periods of ISO fire exposure at which the wall will achieve the above limiting values of relative displacement. This has been calculated by analysing the walls as cantilevered members, but without lateral load. That is, it has been assumed that the lateral restraint or connection offers no resistance to outwards relative displacement, up to the time that the limiting value is reached. A lateral restraint is then applied to the top of the wall to ensure that with further heating, no further relative displacement occurs; (ie. the assumed connection has the force-displacement characteristic shown in Fig. 4.4 below). The analysis is stopped when the lateral force overcomes the moment capacity at the base of the wall - ie. the wall forms a hinge at it's base.

In practice, the analysis was undertaken by applying a lateral force at the top of the wall to achieve (as near as possible) the limiting value of lateral displacement.

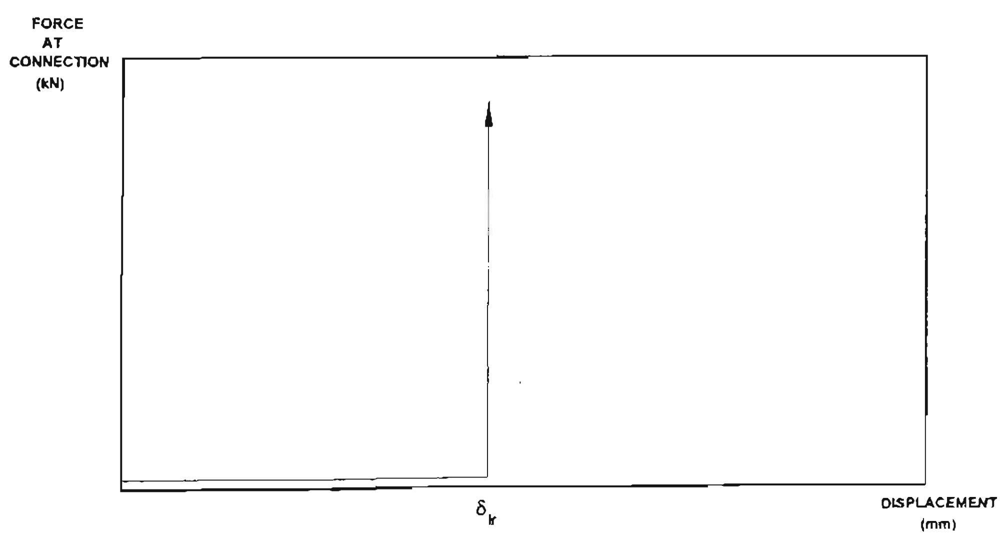


Figure 4.4 Idealised Connection Characteristics

4.2.2.2 Analysis Results and Discussion

The results of the above analysis of cantilevered walls are summarised in Table 4.7. The walls were assumed to be exposed to the ISO fire over their entire height and incorporate a minimum area (225 mm^2/m) of reinforcement. It is also assumed that the connection to the wall from the frame is at the top of the wall.

Table 4.7 Lateral Load Required to Maintain Critical Wall Deformation to δ_{lr}.

Wall Height, (m)				
SFT Exposure Duration; (min)	6		8	
	Limiting δ_{lr} = 240 mm		Limiting δ_{lr} = 320 mm	
	Lateral Load, (kN)	Associated Δ, (mm)	Lateral Load, (kN)	Associated Δ, (mm)
7.5	0.00	306	0.00	600
7.5	0.75	251	1.25	355
10	*Plastic Hinge at Base*	*Plastic Hinge at Base*	*Plastic Hinge at Base*	*Plastic Hinge at Base*

From the above table it can be seen that with no lateral force applied to the top of the 6m wall, the lateral displacement after 7.5 minutes of SFT exposure was 306 mm. The wall was then analysed at this SFT exposure time with varying levels of lateral load with the solution being the one that limited the wall displacement closest to that of the limiting δ_{lr}. For the 6 m vertical wall panel to maintain the limiting displacement, δ_{lr}, of 240 mm after 7.5 minutes of SFT exposure, a lateral load of approximately 0.75 kN must be applied at the top of the wall.

Similarly, for the 8 m vertical wall panel to maintain the limiting displacement, δ_{lr}, of 320 mm a lateral load of 1.25 kN must be applied by the supporting structure. The maximum horizontal force applied to the top of the wall will only reduce the lateral displacements due to "elastic" behaviour by 18% of the total thermal displacement for the 6m case and 40% for the 8m case.

The results summarised in Table 4.7 illustrate a number of aspects of behaviour:

- for the uniform heating situation assumed, the restrained 6 m and 8 m walls achieved the limiting displacements proposed in Equation 4.1 in less than 10 minutes of standard fire exposure. It is unlikely that the supporting frame will have undergone substantial deformation when exposed to this duration of fire exposure. If the frame was subject to the ISO fire it would generally take between 20 - 30 minutes of exposure to obtain very large displacements, as steel temperatures in excess of 750 °C would be expected[2]. However, the nominated limiting displacements (even allowing for 30 minutes of exposure) are the correct order of magnitude. The calculated displacements are likely to be larger than those experienced in practice due to the modelling assumption of one-way curvature, and the assumption of uniform heating up the height of the wall.

- the application of force to limit the displacement to that given in Equation 4.1 resulted in the bending moments at the base of the wall becoming equal to the wall's bending capacity. That is, a "plastic hinge" at the base of the wall was formed.

- the level of force required to achieve a plastic hinge at the base of the wall is very low - 0.75 kN for the 6 m wall and 1.25 kN for the 8 m wall.

4.2.2.3 Simplified Equilibrium Equations

Simplified expressions can be derived for the force likely to be developed in the connection between concrete panels and the supporting steel portal frame. According to O'Meagher et al. (1994), if it is assumed that the maximum relative displacement between panels and supporting frame at the

---

2 Portal frame structures are designed to resist dead and wind loads. Wind loading predominates but, in fire, the high presence of significant wind loads is not considered. Therefore the level of loading associated with a steel frame in a fire is that associated with dead loading. This is relatively low, and when given in terms of Section 12 of AS 4100, a load ratio of less than 0.15. This load ratio translates to a temperature of $905-(690 \times 0.15) \approx 800°C$. Thermally induced forces and moments will reduce this limiting temperature. Portal frame structures are constructed from members with a relatively low exposed surface area to mass ratio ($k_{sm} \leq 25$). According to AS 4100, it would require fire exposure periods of up to 25 minutes to achieve a limiting temperature of 750 °C.

height of the connection is δ_{lr}, and that this relative displacement can be tolerated by the connection system, then the maximum force at the connection can be estimated from:

$$F = \frac{\left(M_b + \sum W_i \cdot \frac{\partial_{lr}}{2}\right)}{H_c} \qquad \text{Eq. 4.3}$$

where:

W	is the weight of the wall
δ_{lr}	is the associated lateral displacement located at the top of the wall
M_b	is the wall's ultimate moment capacity at the base
H_c	is the distance from the connection at the top of the wall to the base of the wall
F	is the lateral force induced at the connection at the top of the wall

The basis for the above expression is now explained. At the point in time when the steel frame undergoes gross deformation, one side of the frame will attempt to pull the outwardly deforming panel inwards. It is at this point that the maximum forces will be experienced by the connection between the frame and the panels. The connection must overcome the base moment capacity (M_b) of the wall *and* the P-δ displacements (see Figure 4.5). The above expression is conservative as in reality it overestimates the force at the connection. It assumes that the application of F does not reduce the outwards displacement δ due to "elastic" deformations. As shown in the previous section, "elastic" deformations of up to 40% of the total lateral displacement can be achieved with the application of a force capable of achieving the moment capacity at the base of the wall.

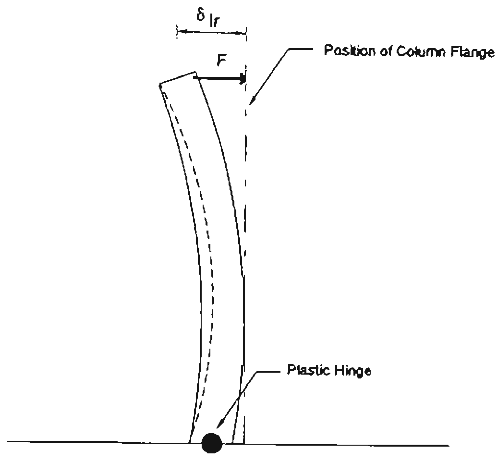


Figure 4.5 Force at Connection where relative displacement is δ_{lr}

Nevertheless, provided a correct value for maximum relative displacement is assumed, the above expression gives a conservative and simple formula for obtaining the connection force.

It is of importance to consider what will happen if the wall is heated to a greater level than that required to achieve the above levels of limiting relative displacement?

The resulting situation is illustrated in Figure 4.6 and is best understood by comparing the behaviour of the wall with and without an adjacent column. If no column is adjacent to the wall, the application of additional heating will simply result in the wall bowing inwards (see Figure 4.6(a)). However, where a column is present this is not possible, and a hinge must develop higher up the wall. This is illustrated in Figure 4.6(b). It is assumed that the cracking moment of the wall is less than it's ultimate moment capacity. This is generally the case.

Another possibility is that the action of prying between the column flange and the base of the wall may result in shear failure at the base of the wall. This is particularly a possibility where the walls are connected to the footing by dowels, as is commonly the case.

Alternatively, depending on it's geometry and construction, the connection between panel and frame may plastically deform.

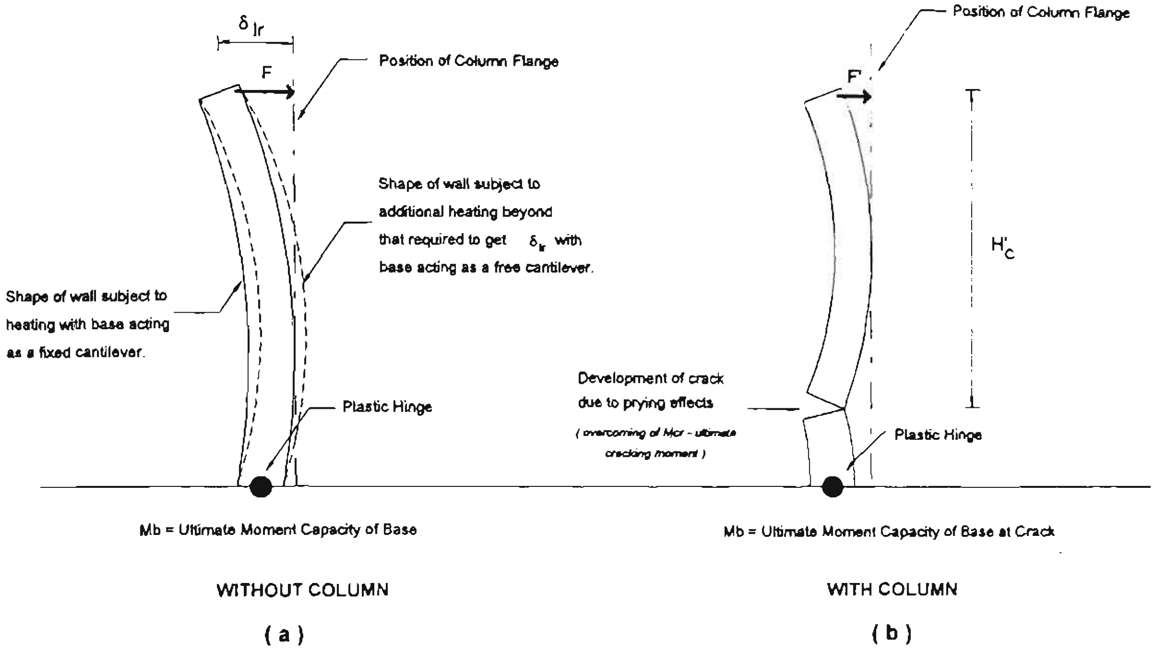


Figure 4.6 Wall Deformation Effects With and Without Supporting Structure

In the case shown in Figure 4.5 (b), the maximum force at the connection will be given by F' ;

$$F' = \frac{\left(M_b + \sum W_i \cdot \frac{\delta}{2}\right)}{H'_c} \qquad \text{Eq. 4.4}$$

Clearly as H'_c becomes less, the force at the connection becomes larger. However, considerable additional relative lateral displacement can be achieved by the formation of a hinge close to the bottom of the wall. In this case the increase in force to F' will not be large.

4.3 Horizontal Panels

4.3.1 Wall Deformation

4.3.1.1 Introductory Concepts

In the case of horizontal wall panels, the panels will bow vertically between the supporting connections, and horizontally between the supporting columns - see Figure 4.7.

For analysis purposes, only the effects of differential temperatures across the panel were considered. "P-δ" effects due to self weight were ignored as horizontal panels are generally less than 2 m in height.

Panels with minimum reinforcement only, (ie. those designed in accordance with AS 3600 - Section 11), are considered for analysis purposes.

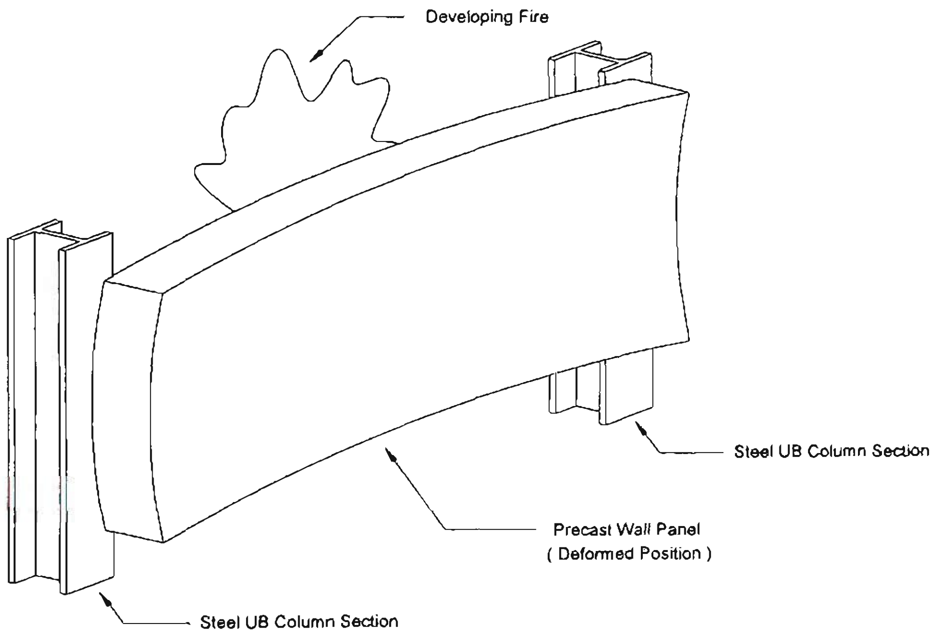


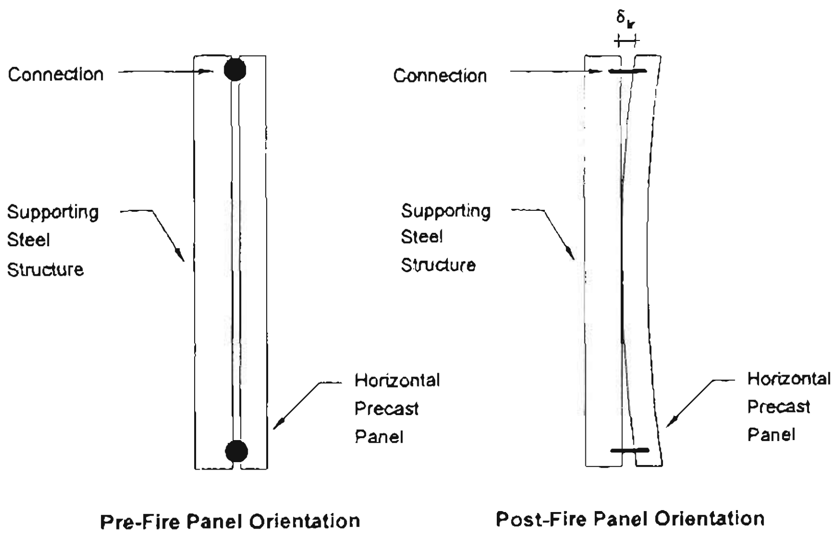
Figure 4.7 Typical Horizontal Wall Panel Orientation and Deformation Characteristics

The relevant details for the wall panels considered are:

Wall Heights	1 m, 1.5 m, and 2 m
Wall Width	6 m
Wall Thickness	150 mm
Concrete Grade, f'_c	32 MPa
Reinforcing Steel	1) Minimum Reinforcement;
	Reinforcement Ratio, p_w = 0.0015 %
	≅225 mm^2/m
	(as per AS 3600 Section 11).
Steel Yield Stress, f_{sy}	450 MPa
End Conditions	Free at the top
	Free at the bottom

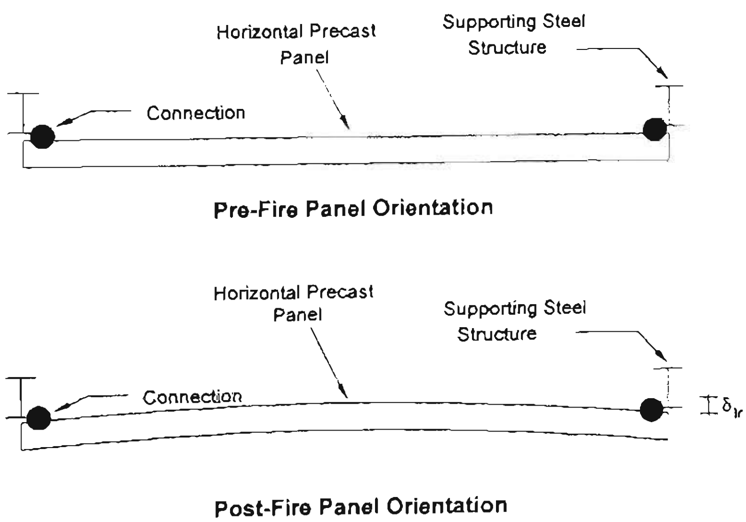
As with the vertical panel case, the horizontal wall panels were analysed with varying levels of standard fire exposure, with periods ranging from 2.5 to 90 minutes duration.

Variation in fire temperature across or along the panel was not considered but the panel was assumed to be subject to uniform fire temperatures. Bending in both directions, as shown below, was considered by assuming the member to bow as a one-way member in either direction.



CASE A Wall Deformation Between Supports - Vertically

Figure 4.8 Horizontal Wall Panel as Analysed in the Vertical Plane



CASE B Wall Deformation Between Supports - Horizontally

Figure 4.9 Horizontal Wall Panel as Analysed in the Horizontal Plane

Case (A) shows the horizontal panel in the vertical plane. The panels were assumed to be supported by four connections located at each of the panel corners. The effects of prying between the typical clip connection, used in fixing such panels to structural steelwork, and the concrete precast panel can be investigated through the analysis of this case.

Case (B) shows the wall as analysed in the horizontal plane. This analysis gives the maximum central displacement as well as the rotation at the end of the panel; ie. the rotation imposed on the connection by the panel rotation.

As for the vertical panel case, the connections must be adequately designed to cater for the relative displacement and rotation arising from panel deformation. As noted previously, values for limiting relative displacement at the connection have been proposed by O'Meagher et al. (1994), and for horizontal panels are:

$$\delta_{lr} = 200\left(\frac{H_f}{10}\right)^2 \qquad \text{Eq. 4.5}$$

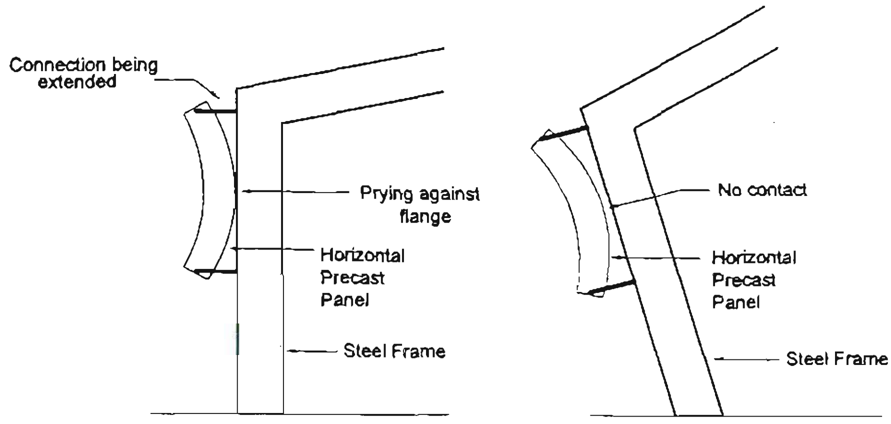
where Hf / 2 is less than 5 m.

$$\delta_{lr} = 200\left(\frac{H_f}{10}\right) \qquad \text{Eq. 4.6}$$

where Hf / 2 is greater than or equal to 5 m.

For the 1m, 1.5m and 2m high panels, the relative limiting displacements, according to the above formulae, are 2 mm, 4.5 mm and 8 mm respectively.

The interaction of the clips with the deforming panel is now considered.



Stage 1 Stage 2

Figure 4.10 Interaction Between Deforming Horizontal Panel and Steel Frame

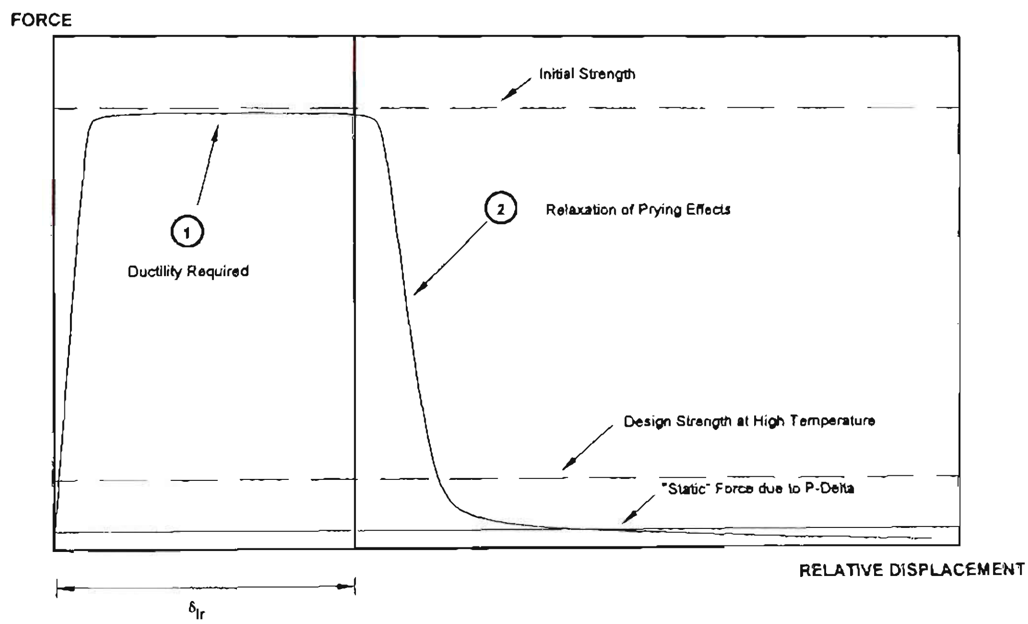


Figure 4.11 Horizontal Panel Connection Characteristics

Two stages of behaviour may be identified when the horizontal clip plate connection is subject to elevated temperatures (see Figure 4.10). Stage 1 is during the initial stages of heating where the panel bends and pries against the column flange. This generates substantial force at the connections. Stage 2 is reached when the frame deforms with the column moving outwards. Additional forces are applied to the connections as a result of the "P-δ" effects (see Figure 4.11) such that the panel moves away from the flange and the prying forces are relieved.

The clip plate connection must possess significant strength and ductility during both stages of deformation.

4.3.1.2 Analysis Results and Discussion

The various horizontal wall panels described in the previous section have been analysed and the results are given Tables 4.8 - 4.11.

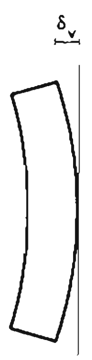


Table 4.8 Displacements of Horizontal Wall Panel in the Vertical Plane

Wall Height;	**Duration of ISO Fire Exposure; (min)**							
(m)	***5***	***10***	***15***	***20***	***25***	***30***	***60***	***90***
1.0	5	5	5	6	7	7	8	9
1.5	5	8	10	11	11	13	15	18
2.0	7	13	16	19	21	22	29	32

Table 4.9 Displacements of Horizontal Wall Panel in the Horizontal Plane

Wall Width;	**Duration of ISO Fire Exposure; (min)**							
(m)	***5***	***10***	***15***	***20***	***25***	***30***	***60***	***90***
6	54	101	133	147	154	173	231	266
8	88	179	245	282	316	334	456	494
10	134	281	399	487	561	609	752	838

Table 4.10 Rotations of Horizontal Wall Panel in the Horizontal Plane (Degrees)

Wall Width; (m)	Duration of ISO Fire Exposure; (min)							
	5	10	15	20	25	30	60	90
6	1.6	3.3	4.5	5.0	5.3	5.9	8.1	9.3
8	2.1	4.6	6.4	7.4	8.4	8.8	12.1	13.2
10	2.7	5.9	8.5	10.4	12.0	13.1	16.2	18.0

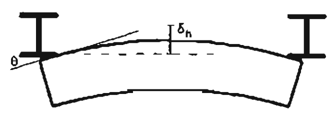


Table 4.11 Displacements at the Connection Associated with Rotations in Table 4.9

Wall Width; (m)	Duration of ISO Fire Exposure; (min)							
	5	10	15	20	25	30	60	90
6	2	3	5	5	6	6	9	10
8	2	5	7	8	9	9	13	14
10	3	6	9	11	13	14	17	19

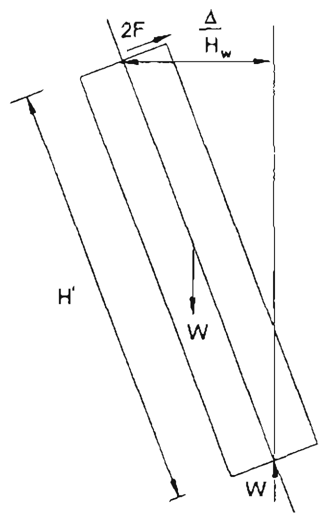
From Table 4.8, it can be seen that for the horizontal panels when analysed in the vertical plane, the limiting displacements at either edge of the panel vary with panel height as well as with ISO fire exposure level. By increasing the horizontal wall panel height from 1m to 1.5m the relative displacement doubled. By increasing the horizontal wall panel height from 1m to 2m the relative displacement tripled. These results show that the height of the wall panels affects the level of relative displacement for all of the ISO fire exposure levels analysed.

Table 4.9 shows the effect of increasing the wall width from 6m to both 8m and 10m. The displacements are not greatly affected at the lower levels of ISO fire exposure but with higher exposure levels the displacements change quite drastically. By changing the effective wall width from 6m to 8m the change in relative displacement is of the order of 1.5 - 2 times the 6m displacements. By changing the effective wall width from 6m to 10m the change in relative displacement over all fire exposures is of the order of 2.5 - 3.2 times the 6m displacements.

Table 4.10 gives the rotations associated with the horizontal bending of the panels. This is quite useful as it allows the total relative displacement that must be accommodated by the clip connection to be determined. Table 4.11 gives the relative displacements associated with rotation at the connections. The total values of relative displacement that the connection must withstand are equal to the sum of those given in Table 4.8 and 4.11. It should be noted that these total values of relative displacement are four times greater than those given by Equation 4.5 at a fire-resistance level of 20 - 30 minutes. The calculated values are likely to overestimate the actual displacements due to the conservative nature of the model (one-way bending assumed in both directions) and the assumption of uniform heating of the panel. Despite this, however, it would appear that the total relative displacements may well be greater than those given by Equations 4.5 and 4.6.

4.3.1.3 Simplified Equilibrium Equations

Assuming that the prying effects have been relieved, as discussed in Section 4.3.1.1, it is necessary for the connections to resist the forces resulting from the "P-δ" effects associated with the outwardly deforming column.



Note: 4 connections per Panel; (2 at the top, 2 at the bottom)

Figure 4.12 Deformed Horizontal Wall Panel Orientation

where:

- W is the self-weight of the wall section considered
- F is the force at the top connection

H_w is the height of the wall section considered

Δ / H_w is the displacement of the wall panel expressed as a ratio of the height of the wall section considered.

[*This is really equal to the outwards slope of the portal frame column (see Figure 4.10)*]

Considering one of the panels as a free body, the following equilibrium equation can be written:

$$2F \cdot H' = W \cdot \frac{\Delta}{H_w} \cdot \frac{H'}{2} \qquad \text{Eq. 4.7}$$

where:

Δ is the outwards displacement of the frame

$$F' = W \cdot \frac{\Delta}{4H_w} \qquad \text{Eq. 4.8}$$

The resulting forces are small.

4.4 Conclusions

From the above analyses, many conclusions can be drawn regarding the effect that elevated temperatures have on the displacements between concrete wall panels and the steel supporting structure. These are summarised below.

For both vertical and horizontal panels, attached to a steel frame, the magnitude of relative displacement increased with increasing fire exposure.

For isolated (ie. cantilevered) vertical panels with minimum reinforcement as required by AS 3600, and subject to whole wall SFT exposure, the maximum fire-resistance period achieved was 10 - 30 minutes for a 6m wall and 5 - 10 minutes for an 8m wall. This assumes that wall panels are exposed to uniform heating over their entire internal area. However, in practice, it is considered unlikely that walls will be subject to such uniform heating.

To further investigate the likely influence of non-uniform heating, the behaviour of the wall panels subject to non-uniform heating was examined, and it was shown that the fire-resistance of the walls considered could be increased by up to 50% if the panel was subject to uniform heating over only half the height.

It was found that cantilevered walls will ultimately collapse *outwards* unless restrained by the steel frame.

The behaviour of the typical vertical wall panels when restrained at the top was studied, using the model developed in Chapter 3. It was found that the restraint force reached a value capable of forming a plastic hinge at the base of the wall, after a standard fire exposure period of only 10 minutes for both 6m and 8m walls.

During the stages of a fire, before the frame undergoes gross-deformation, the frame will hold the panel in position. Due to the outwards displacement of the wall there will be some relative displacement between the wall and the frame. Any connection between the panel and the frame must be able to absorb this relative displacement or resist an increasing force as the panel bends against the column flange.

Approximate values have been suggested by O'Meagher et al. (1994) for these relative displacements that may develop between the supporting steel frame and the concrete wall panels during fire; (refer to 4.2.2.1 and 4.3.1.1). For vertical panels these are:

For Vertical Panels;

$$\delta_{lr} = 200 \cdot \left(\frac{H_c}{5}\right)^2 \qquad \text{where } H_c \text{ is less than 5 m.}$$

$$\delta_{lr} = 200 \cdot \left(\frac{H_c}{5}\right)$$ where Hc is greater than or equal to 5 m.

For vertical panels, it was found that the above values of relative lateral displacement were achieved by the restrained panels when subject to full height fire exposure in less than 10 minutes. It is unlikely that significant frame deformation resulting in the walls being pulled over would occur in less than 20 - 30 minutes of standard fire exposure. Nevertheless, the limiting displacements recommended by the above formulae are of the correct order of magnitude. The calculated displacements are likely to be larger than those in practice because of the assumption of single curvature bending and uniform heating over the height of the wall.

If the curvature of the wall is increased to give relative displacements beyond assumed limiting values, it was found that there are several possible outcomes.

(i) The force may build up in the connection to such an extent that the connection fails.

(ii) The force may increase slightly and the connection may deform plastically at high temperatures - thus accommodating more wall deformation. This depends on the type of connection.

(iii) The force may increase slightly (<10%) with formation of a new hinge higher up the panel.

(iv) Due to prying of the concrete panel against the column flange, the panel may fail in shear and move outwards at the base.

O'Meagher et al. (1994) recommends values for relative displacements between supporting columns and clips.

For Horizontal Panels;

$$\delta_{lr} = 200\left(\frac{H_f}{10}\right)^2$$ where Hf / 2 is less than 5 m.

$$\delta_{lr} = 200\left(\frac{H_f}{10}\right)$$ where Hf / 2 is greater than or equal to 5 m.

From the analysis, assuming full fire exposure, it appears that the displacements may be up to four times the value given by the above formula.

The interaction of horizontal panels with the supporting column was described and a simplified expression for connection force derived.

CHAPTER 5 ASPECTS OF CONNECTION BEHAVIOUR

5.1 Introduction

The previous chapter considered the deformation and strength requirements for connections between concrete wall panels and the supporting structure.

Under real fire conditions, the unprotected steel connections will become hot and their strength may be significantly reduced. As highlighted by Rubert and Schaumann (1988) and Kirby and Preston (1988), the critical temperature range where the steel section experiences loss in strength in real fire situations is between 600 °C and 800 °C. At these temperatures, the strength of the connections will be significantly reduced and time-dependent deformation (thermal creep) may be significant. These aspects are considered in this chapter in relation to the most common form of panel connection.

5.2 Connection Situations Considered

Common situations utilising steel clips as connections have been considered in earlier chapters and are again shown in Figure 5.1.

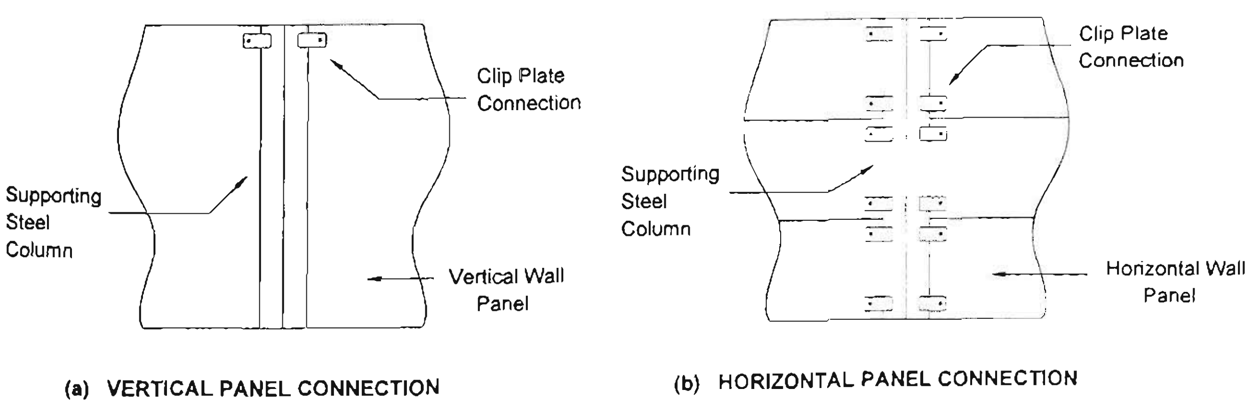
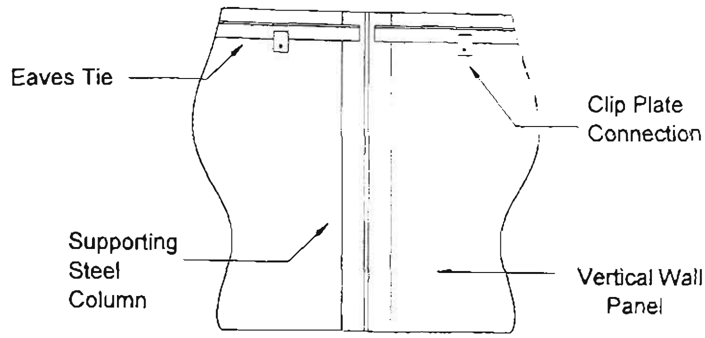


Figure 5.1 Common Forms of Connections that Utilise Steel Clips



(c) EAVES TIE CONNECTION

Figure 5.1 (continued) Common Forms of Connections that Utilise Steel Clips

Some initial comments on the situations shown in the above figure are warranted.

As discussed in the previous chapter, a vertical panel adjacent to a column section, will bow away from the column under fire exposure and for cases (a) and (c), substantial relative displacements between wall panel and column must be accommodated. In situation (a), it would appear that it is unlikely that the clip will offer adequate strength and deformation capacity. In situation (c), the expansion of the eaves tie may provide sufficient flexibility to allow the panel to bow away from the column due to expansion of the eaves tie and consequent buckling (see Figure 5.2); but will the clip plate connection have sufficient strength to hold the panel in position and then later participate in pulling the panel inwards with the frame?

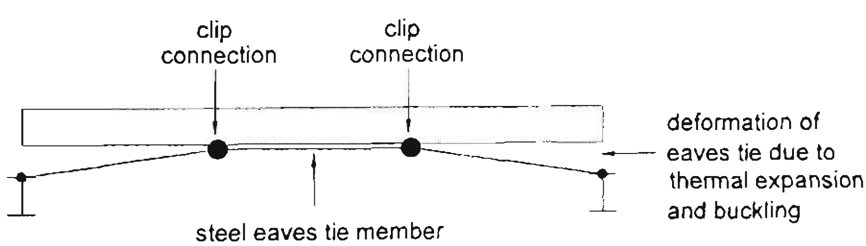


Figure 5.2 System Capable of Minimising Prying Effects

Even for the horizontal panel shown in Figure 5.1(b), the clips will be deformed plastically due to bowing of the panel and prying against the column flange (see Chapter 4).

The clip connection shown schematically in Figure 5.3 is representative of most clips used in this form of construction, and is therefore worthy of investigation.

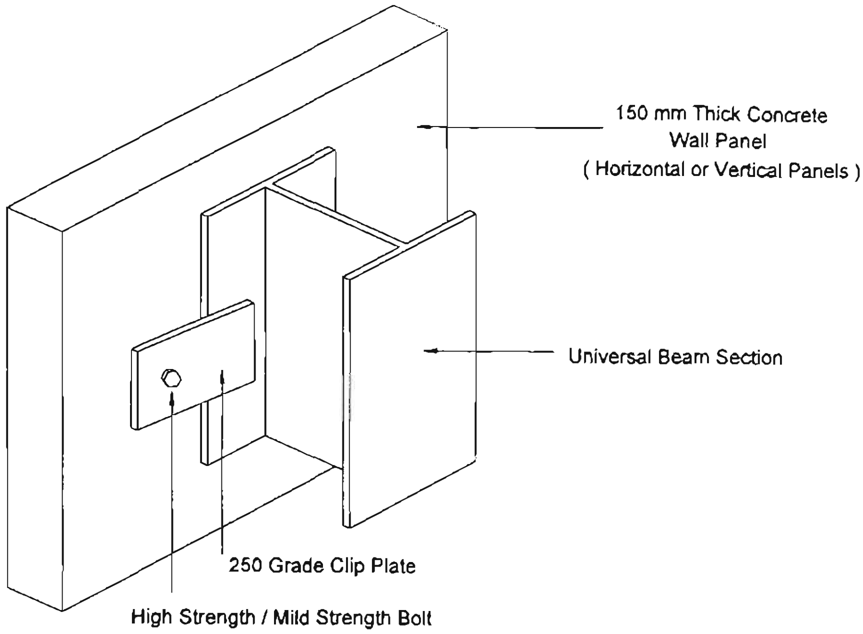


Figure 5.3 Typical Clip Plate Bearing Connection Used in Experimental Testing

In practice, the clip plate may or may not be welded to the column flange (or eaves tie in the case of Figure 5.1(c)). The behaviour of both welded and non-welded clips are considered in this chapter.

The clip plate is generally of Grade 250 steel and has a length of 150 - 200 mm and a width of 100 - 150mm.

This type of connection generally utilises Grade 4.6 or Grade 8.8 bolts which are incorporated within either 16 mm or 20 mm diameter ferrules. The ferrules used for the testing program utilised Grade 8.8 bolts. It is important to note that sometimes expansion anchors are used in favour of the ferrule connection.

The advantage of using the clip plate connection is that it is economical, it allows rapid erection and fixing of the panels, and it does not require precise fit-up during construction.

5.3 Method of Approach

In order to study the behaviour of this type of connection, a series of isolated connection tests were performed at ambient and elevated temperature conditions.

Eight tests were conducted with four tests being at ambient temperature and four at elevated temperatures.

Ambient temperature tests were conducted to provide a basis for comparison with the behaviour under elevated temperature conditions as well as to obtain the ambient temperature strength.

Also investigated in the series of tests was the influence of creep on the behaviour of the connection at elevated temperatures. The elevated temperature tests were conducted at temperatures of 600 °C and 800 °C.

A summary of the testing program is shown in Table 5.1.

Table 5.1 Summary of Testing Program

Test Number (#)	**Clip Welded ?** (Yes / No)	**Clip Temperature** (°C)	**Test Designation**
1	No	Ambient	NW20-1
2	No	Ambient	NW20-2
3	No	600	NW600
4	No	800	NW800
5	Yes	Ambient	W20-1
6	Yes	Ambient	W20-2
7	Yes	600	W600
8	Yes	800	W800

All of the steel used in the construction and assembly of the connections was taken from the same section of plate. This was done so that there would be minimal variance of the tensile properties between the eight connections.

5.4 Test Set-Ups

The test specimens incorporated four identical concrete slabs with which the eight tests were carried out. For some of the elevated temperature tests the slabs were able to be re-used as there was no visible structural damage. The various structural components that were incorporated into the test specimens are described in Table 5.2 below, (see Figure 5.4).

Table 5.2 Description of Test Samples

COMPONENT	DESCRIPTION	ASPECT	CLASSIFICATION *Nominal*	CLASSIFICATION *Measured*
CONCRETE	Slab	Width	650 mm	649.75
		Length	650 mm	650.50
		Depth	650 mm	500.00
		28 Day Strength	32 MPa	46 MPa
		Reinforcement	F82 Mesh	F82 Mesh
STEEL	Beam	Designation	410 UB 60	410 UB 60
	Plate	Size	150 x 100 x 10	150 x 100 x 10
		Grade	250 MPa	280 MPa
	Bolts	Size	M16	M16
		Grade	8.8	8.8
	Ferrules	Size	M16	M16
		Reinforcement	300 mm long Y12 Bar	300 mm long Y12 Bar
		Grade	8.8	8.8
	Weld	Type	E41XX - 6mm SP CFW	E41XX - 6mm SP CFW
		Size	2 / 50 mm	2 / 50 mm

Figure 5.4 gives a schematic description of the specimen tested.

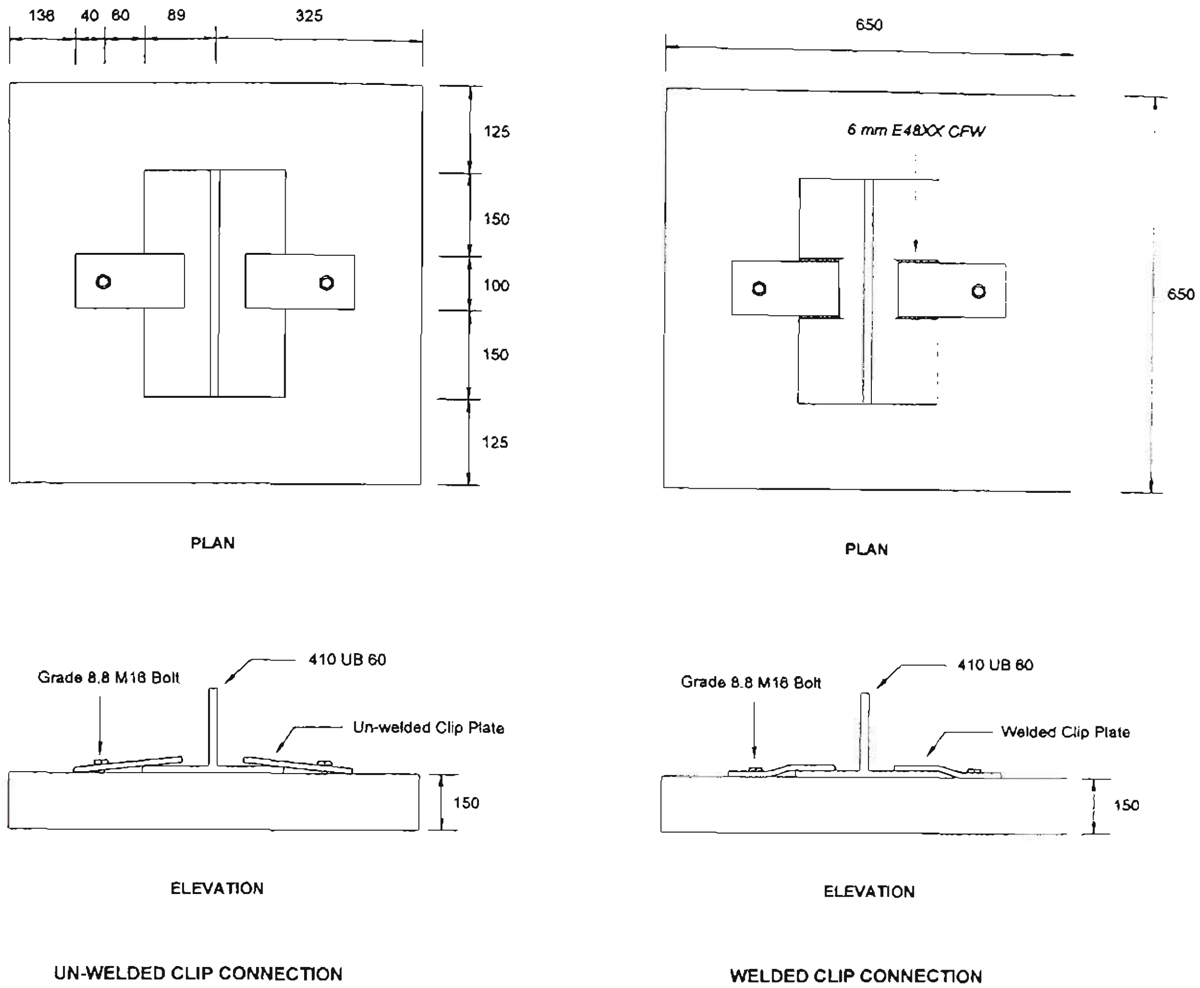


Figure 5.4 Details of Connection Samples Tested

In addition to the full scale connection tests, the stress-strain characteristics of the concrete used for the slab was determined (Appendix E), as was the 28 day compressive strength. The yield and ultimate strength of the steel plate used for the clips was also determined (Appendix E).

5.4.1 Construction of Slab Forms

The forms for the concrete slabs were constructed from steel. A single layer of F82 mesh was located centrally, ie. equal concrete cover top and bottom, in the formwork and spaced to give adequate cover using four bar chairs situated symmetrically within the slab. The ferrules had to be embedded into the concrete with the right tolerances and this was achieved by supporting the ferrules off an angle section that spanned the width of the formwork, as shown in Figure 5.5.

Figure 5.5 Formwork and Reinforcement

Five cylinders were taken during the concrete pour and these were progressively tested to enable the determination of the compressive strength of the concrete at any given time prior to testing. Some of the cylinders remained with the test slabs, which were allowed 28 days to cure, and one of these was tested just prior to the conduct of the connection tests. The results of the compressive tests are given in Appendix E.

While the slabs were curing, the other test components were fabricated.

5.4.2 Assembly of Test Specimens

Assembly of the test specimens was undertaken within the testing apparatus.

The underside of the concrete slab was supported on two sections of universal beam which were firmly and rigidly connected to the loading floor beams as shown in Figure 5.6. The concrete slab was then bolted with eight Grade 8.8 bolts to the beams by means of a steel restraining frame fabricated from angles. This provided two-way restraint to the slab throughout the testing procedure, reducing torsional and bending effects within the slab.

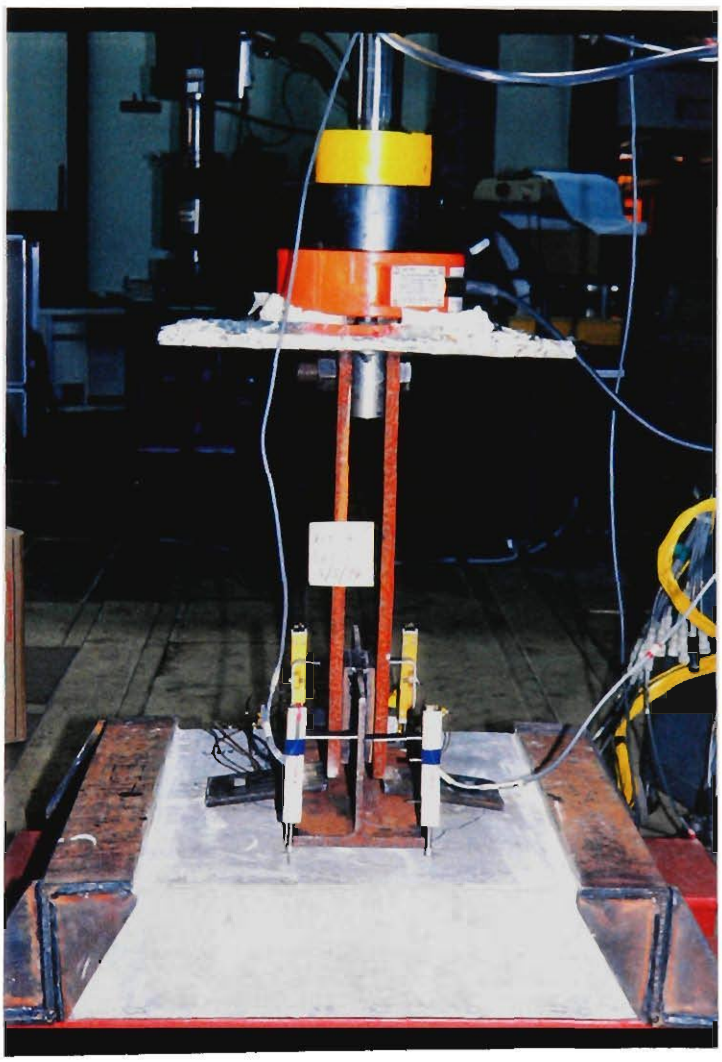


Figure 5.6 Test Apparatus

An added step in making sure that the slab was bearing evenly on the beam support was to pack the area between the slab and the restraint with cornice plaster.

The restraint limited the effects of torsion and bending on the slab and effectively held the system rigid over all four edges of support.

Load was applied to the specimen by a tension jack via two plates connected to the web of the 460UB "T" section; (see Figure 5.6). The plates were bolted at one end to the web of the beam and to the loading ram at the other.

Certain differences existed in the assembly of the steel components of the non-welded and welded test samples. The differences are highlighted below.

5.4.2.1 Non-Welded Connection

In the non-welded case, the beam section was placed in the centre of the slab between the cast-in ferrules. The clip plate, one on each side of the web, was bolted to the flange of the beam using a single M16 Grade 8.8 bolt. The bolt was snug tightened to a torque setting of 400 Nmm as required by AS 4100.

The clip plates used in the non-welded case were straight and only had one point of contact with the beam flange, as shown in Figure 5.4.

5.4.2.2 Welded Connection

The welded connection was set-up in a similar manner to that described above with the beam being situated between the cast-in ferrules. The main difference was that in this case, the clip plate was purposely bent so that the plate had two points of contact; ie. along the slab and along the beam flange. This was done to facilitate welding.

The bearing plate was welded along the two edges that run perpendicular to the beam's flange with a continuous 8mm fillet weld; (see Figure 5.4).

5.4.3 Instrumentation of Test Samples

In order to study the behaviour of the connection at elevated temperatures, the samples were instrumented with thermocouples (Type-K) to measure temperature, and linear potentiometers to measure displacements.

The potentiometers were of two types; 30 mm and 100 mm travel lengths. Depending on the positioning of the linear potentiometers, different travel lengths were used.

For the elevated temperature tests, fourteen thermocouples were placed under the clip plate and on the surface of the web of the beam. This enabled the constant monitoring of temperatures throughout the initial heating period and for the duration of the test.

Four linear potentiometers were used to measure the displacements at various times throughout the test. The set-up is shown in Figure 5.6. Axial displacement of the bolts was also measured, for the ambient temperature tests, to measure the extent of elongation as well as any movement of the ferrule anchorage. For the elevated temperature tests, it was not possible to measure the elongation of the bolt due to the direct heating of the plates and bolts with the gas torches.

The output from all instruments was measured using a PC-based data logging system.

For the ambient temperature tests, for both the welded and non-welded connections, the clip plate was covered with a layer of resin. This was done so that as the plate deformed, and opened up under load, there would be a clear indication of the onset of yielding.

5.5 Test Procedure

The forces were applied to the specimens using the jacking system shown in Figure 5.6. This consisted of a single tension jack of 200 kN capacity, and load cell with the facility to control the rate of ram movement. The jacking system was attached to a cross-head frame and mounted on the reaction floor at the Melbourne Laboratories of BHP Research.

The connections were tested under both load and position control.

The ambient temperature tests were all conducted under position control. In this case, the force applied at the connection was governed by the displacement of the loading ram. The ram displacement was able to be controlled to a given pre-determined rate.

The elevated temperature tests, on the other hand, were tested under load control. This meant that the loading of the connection was governed by incrementing the load at pre-determined times during the test. This enabled the effects of connection creep to be studied. Details of the load and temperature histories are given in Sections 5.6.3 and 5.6.4 for the non-welded and welded tests, respectively.

The heating of the connections was facilitated using two gas torches, one on either side of the beam web as shown in Figure 5.7.

Figure 5.7 Heating Up of Test Samples

This method of heating enabled the plate temperatures to be regulated to an accuracy of ±30 °C, as shown in Figure 5.8.

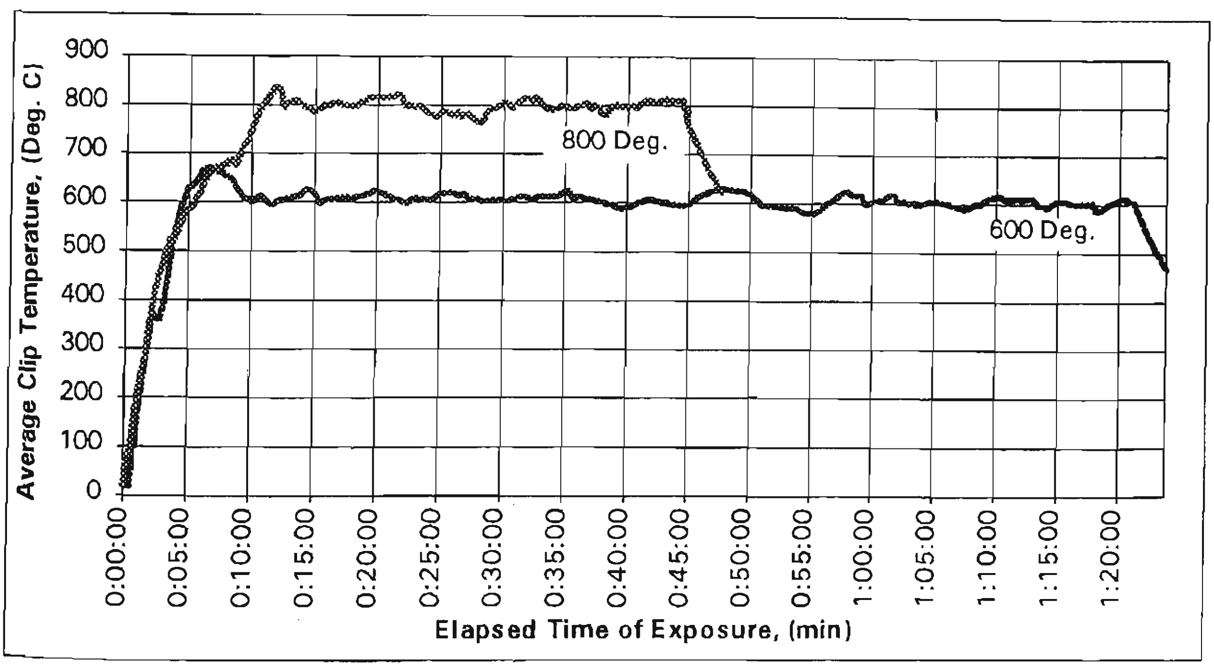


Figure 5.8 Maintenance of Clip Plate Temperature Over Time

The gas torches were used to slowly and uniformly heat up the clip plate to a specified temperature of either 600 °C or 800 °C, depending on the test. Once the target temperature was attained on both of the connections, and seen to be adequately maintained, the loading of the specimens commenced. The target temperature was held constant throughout the test.

Because the plate was tested as an isolated component, the area surrounding the clip plate had to be adequately protected and insulated from heat. This was achieved using mineral wool (see Figure 5.7). This also served as protection for the instrumentation situated close to the heated clip plate.

5.6 Test Results and Discussion

A summary of the test results, including a description of the failure modes, is given in Table 5.4. Each test is designated with a specific notation.

Table 5.3 Summary of Test Results

Test Number (#)	Test Designation	Maximum Applied Load (kN)	DEFLECTION AT PEAK LOAD (mm)	DESCRIPTION OF FAILURE MODE
1	NW20-1	35.0	27.9	Clips yield & open, Bolts deformed
2	NW20-2	35.0	29.1	Clips yield & open, Bolts deformed
3	NW600	10.5	14.8	Clips yield & open, Bolts deformed
4	NW800	1.5	15.6	Clips yield & open, Bolts deformed
5	W20-1	84.5	11.4	Clips yield, Concrete cone failure
6	W20-2	109.8	17.5	Clips yield, Concrete cone failure
7	W600	55.0	32.7	Clips yield, Bolts deform & fail
8	W800	13.0	36.9	Clips yield, Bolts deform & fail

Each test is now discussed in detail.

5.6.1 Ambient Test / Non-Welded Connection

The two ambient temperature tests of the non-welded clip connection were conducted to provide a basis for comparison with the elevated temperature tests. As noted previously, the test was conducted in position control. The rate at which the loading ram was displaced was 1 mm / min.

The resulting applied load versus displacement relationships are shown in Figure 5.9. It will be noted that the load-displacement relationships for both tests are very similar.

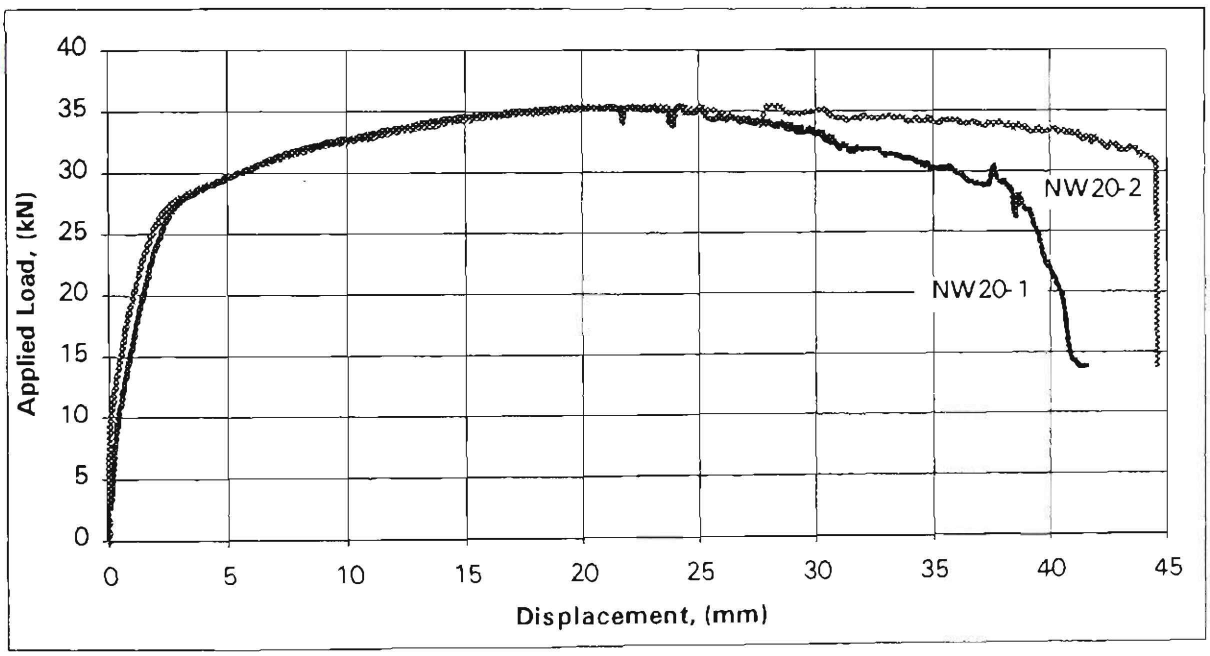


Figure 5.9 Load Displacement Chart for Non-Welded Connections Tested at Ambient Temperature

Under the increasing tensile load, the clip plate was able to resist the force until approximately 27 kN when the plate experienced significant plastic deformation. In both tests, the maximum tensile load that the connection was able to carry was approximately 35 kN, after which the connections started to off-load. The deflection at this point was 25 - 28 mm.

As previously mentioned, a lacquer coating was applied to the top side of the clip plate to highlight the areas on the plate should yielding occur. Most yielding (cracking of the lacquer coating) was found to occur where the plate came into contact with the beam flange as shown in Figure 5.10.

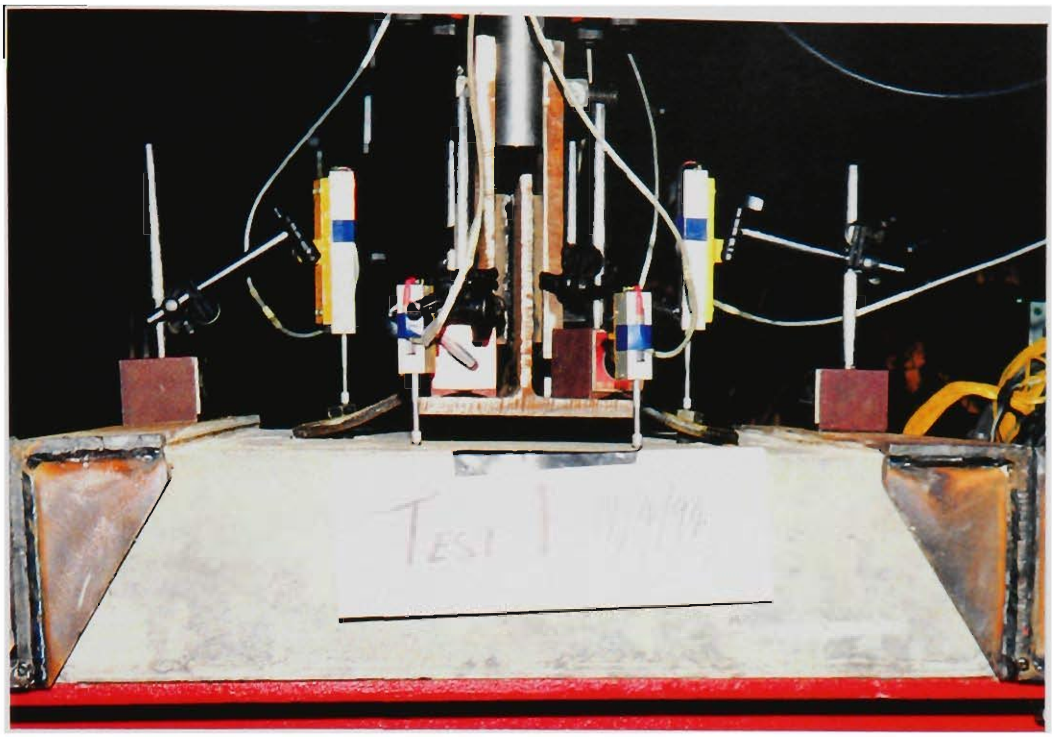


Figure 5.10 Lacquered Clip Plate Under Strain

Eventually the clips opened up to such an extent that the beam pulled away from the slab (Figure 5.11).

Figure 5.11 Failed NW-20 Clip Plates

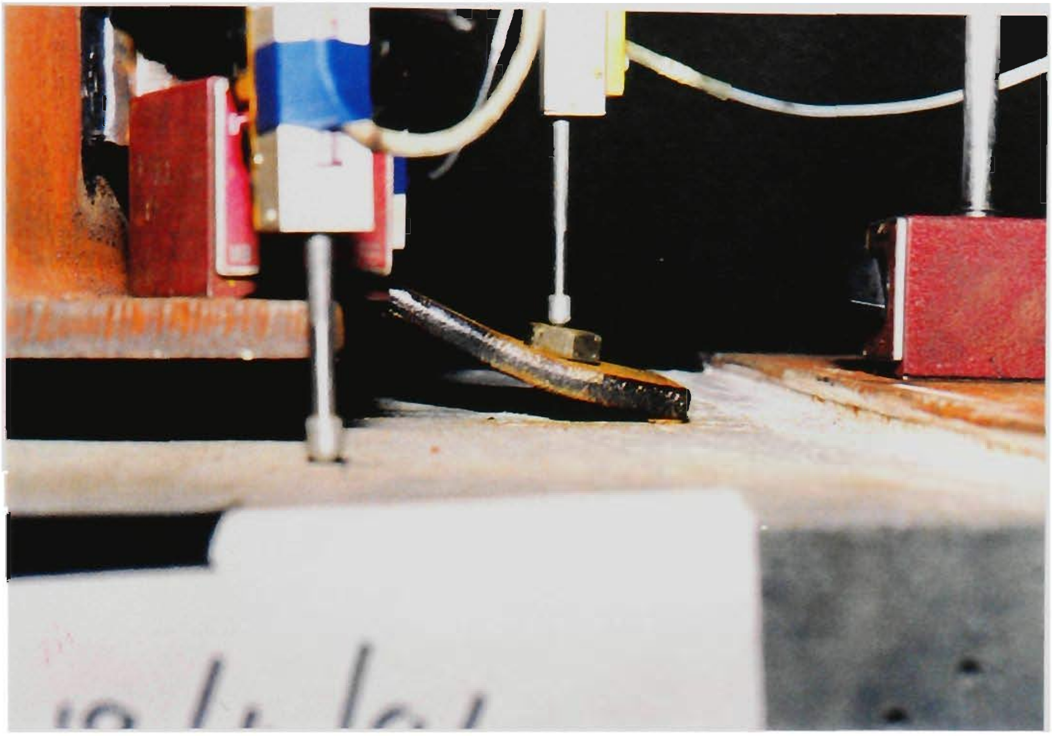


Figure 5.12 Deformation of NW-20 Clip Plate

In neither test was the beam flange damaged. The test slab was observed after the each test and no damage was visible to the ferrule anchorages or the ferrule threads. The bolts were removed after the test and measured for deformation and elongation (see Figure 5.13).

Figure 5.13 Bolt Elongation And Deformation of Test NW-20

5.6.2 Ambient Test / Welded Connection

This test was also conducted under position control. The difference here was that the clip plate was welded on the two sides that made contact with the beam flange, (see Figure 5.14).

Figure 5.14 W-20 Test Clip Plate Under Strain

The load-displacement characteristics for tests 5 and 6 are given in Figure 5.15, where it can be seen that a different response was obtained for each test. This difference was attributed to the level of restraint afforded by the slab restraining frame.

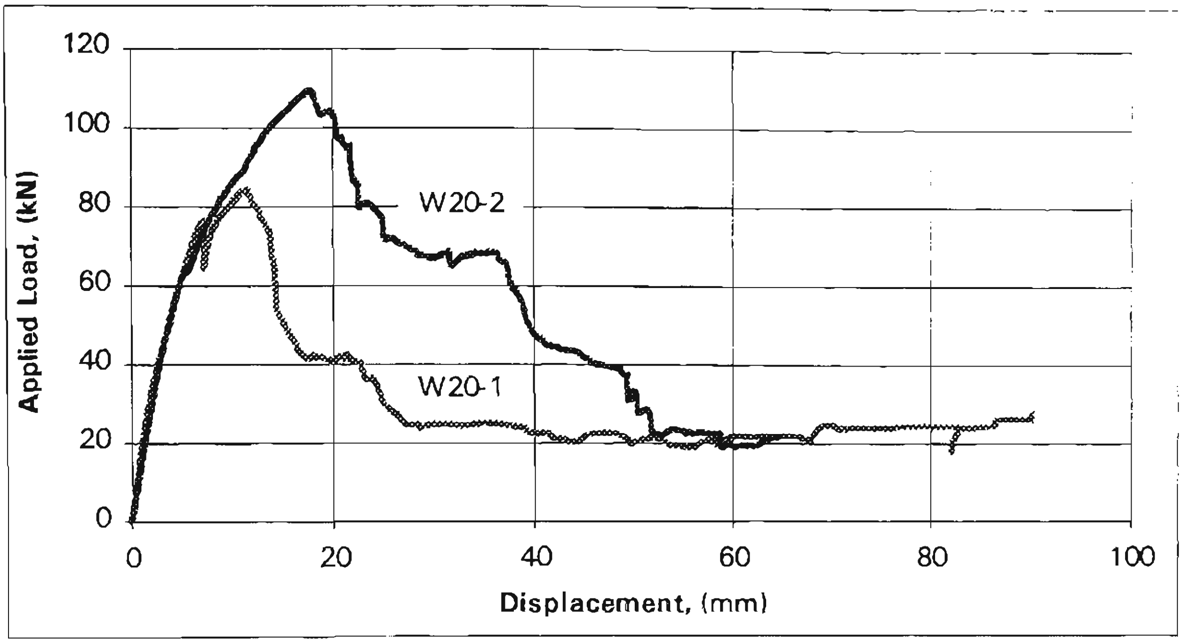


Figure 5.15 Load Displacement Chart for Welded Connections Tested at Ambient Temperature

The first test was carried out with the slab being supported and restrained on two edges, (see Figure 5.16). This set-up was used for the previous tests on the ambient/non-welded connections and gave no indication of affecting the results.

Figure 5.16 Premature Failure of W-20 Clip Plate

Failure in this case occurred at a peak force of just 85 kN with an associated deflection of approximately 11 mm, (see Figure 17). Simple hand calculations, using the models given in Appendix F, to check the capacity of the clip plate indicated that it was likely to have a significantly greater capacity than 85 kN.

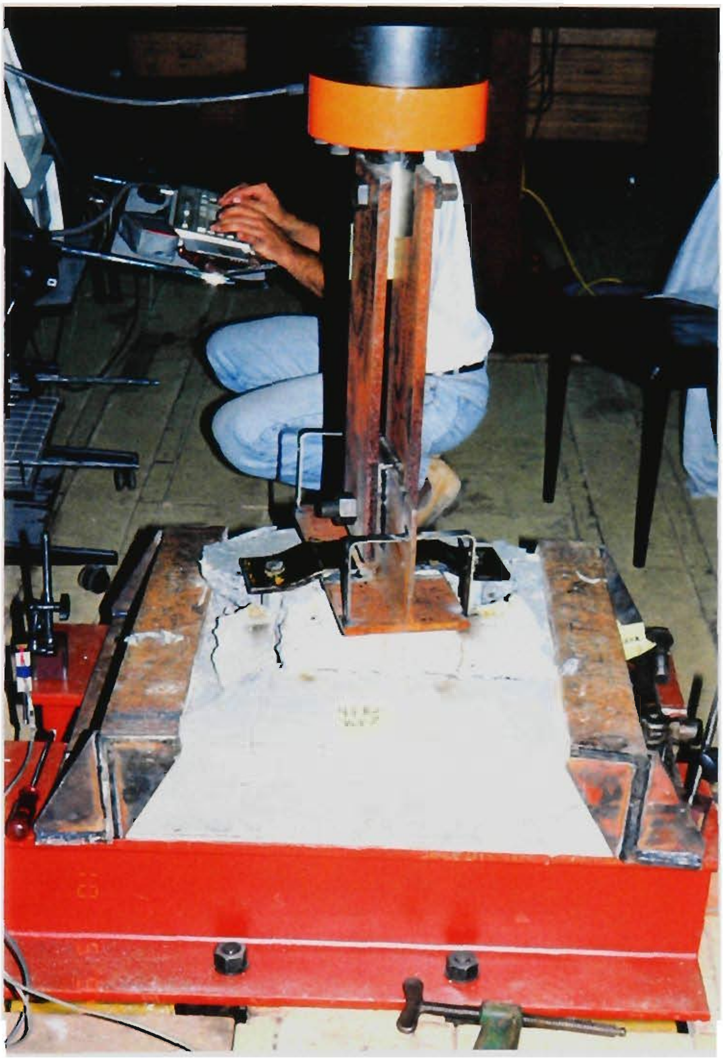


Figure 5.17 Premature Failure of Two-Way Support

Close examination of the slab revealed some flexural cracking and it was found that yielding of the slab reinforcement had occurred between the supported edges directly under the UB section.

For the second test a modification to the test rig was made so that the edges of the test slab were restrained on all four edges. This resulted in the slab resisting the connection forces through two-way bending. This, in practice, will be the situation with a ferrule inserted into a wall.

The peak load experienced by the connections, in W20-2, was approximately 110 kN which occurred at a deflection of approximately 18 mm. This figure compares well with that predicted using simple design models - see Appendix F.

Test W20-2 finally failed by the ferrule pulling out of the concrete (see Figure 5.18). As can be seen from this figure, a large cone of concrete was ripped out of the slab.

Figure 5.18 Proper Failure of W-20 Clip Plate Test

5.6.3 Elevated Test / Non-Welded Connection

The elevated temperature tests of the non-welded clip plate connection were conducted under load control. This allowed the load-deflection relationships, with respect to time, to be attained. This is important in assessing the likely influence of thermal creep on connection behaviour. The load was increased in 0.5 kN increments.

The clip plate temperature-time relationships associated with NW600 and NW800 tests are similar to those shown in Figure 5.8. The beam flange temperature was found to be significantly lower than the clip plate temperature which indicates the occurrence of heat loss from plate to flange.

The displacement-time and applied load-time plots for tests 3 and 4 are given in Figures 5.19 and 5.20, for clip plate temperatures of 600 °C and 800 °C, respectively.

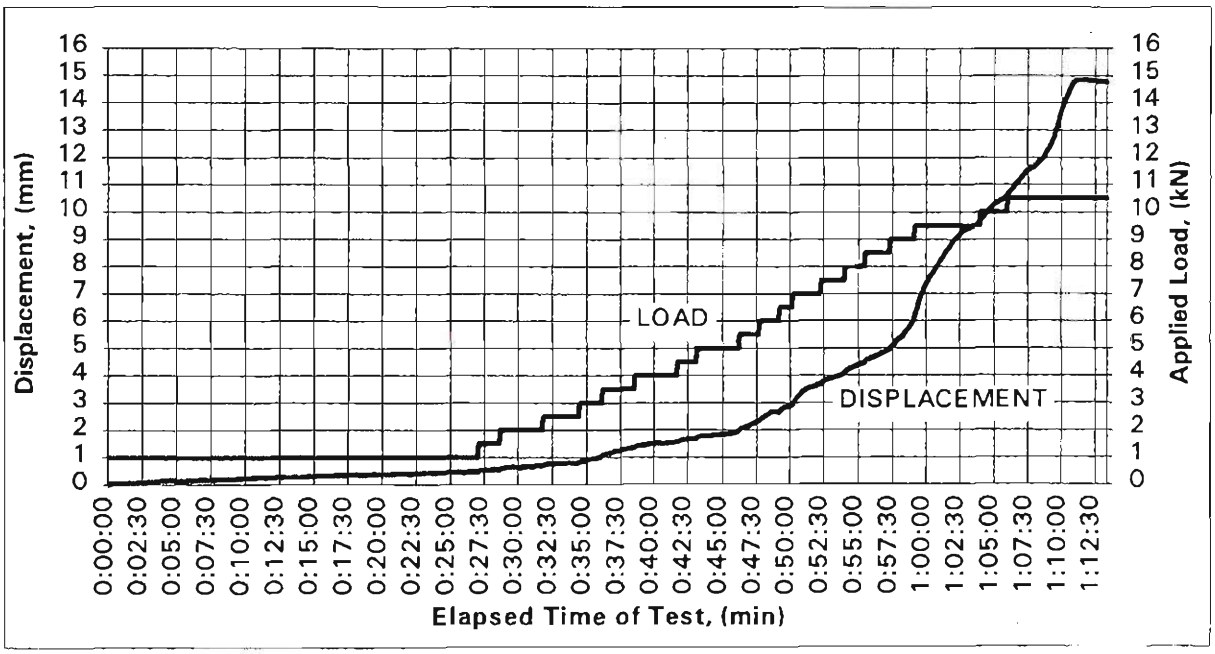


Figure 5.19 Creep Effects for Non-Welded Connection Tested at 600 °C

The time that the load was maintained at any one load setting was in the order of 2-3 minutes. If in this time, no evident creep effects were noticed, then the load was incremented.

The level of stress in a steel member influences the load carrying capacity of the member and is critical at temperatures of 600 °C and greater. Morley and Royles (1979), state that the yield stress of steel is reduced to 60% of the room temperature value at this elevated temperature. AS4100 suggests that the strength is reduced by 60%.

It will be noted that for test NW600, the rate of deformation increases as the load level is increased. This reflects the fact that greater creep deformations will occur for higher load levels. It should also be observed that the clip deformation at which separation of the clip and flange began to occur was approximately 16mm. This was also the case with the ambient temperature tests; NW20-1 and NW20-2.

In real fires, the clips may be subject to elevated temperatures for significant periods of time. Figure 5.19 illustrates that, unless the load level is kept low, significant time-dependent deformations will occur. For example, using the slope of the displacement-time relationship, if the load level required in the real fire situation is 20% of the ambient clip strength, (ie. 7 kN), then failure by clip opening would be expected after approximately 2 hours and 30 minutes. If the load level was 10% of the ambient clip strength, (ie. 3.5 kN), then failure by clip opening would be expected after approximately 4 hours and 30 minutes. On the other hand, if the load level was 5% of the ambient clip strength, opening would be expected after approximately 14 hours at 600 °C.

These values of approximate time to failure are made on the basis of the following assumptions:

(a) that there is a constant deformation rate for specified load level and temperature and,

(b) that there is a unique displacement at which failure occurs.

According the Fields (1989), creep strain varies linearly with temperature up to 600 °C, but at higher temperatures the rate may increase with time. Knight (1972) also makes the assumption of linear variation of creep strain with time. Assumption (a) is considered reasonable for the purpose of the above calculations.

Failure of this connection specimen was directly attributed to the clip plate bending under the increasing load at an elevated temperature. Both the bolt and plate lose strength and stiffness under prolonged exposure at elevated temperatures, thus allowing the connection to deform and bend. The bolts tended to elongate and deform as the plate yielded and opened.

It was recognised that at 800 °C, creep effects would be more significant. Lower levels of load were applied, and at each load level, the load was maintained for generally longer periods.

From Figure 5.20, it can be again seen that as the level of load is increased, the rate of displacement increases. Again, the value of displacement at which the clip plate opened to allow separation of clip and flange was found to be approximately 16 mm. The displacement-time results of Figure 5.20 indicate that at 800 °C, if a load level of 0.85 kN (2.5% of ambient strength) was applied to the clip, then separation would have occurred after approximately 30 minutes. If the load level was increased to 5% of the ambient clip strength then separation would have occurred almost immediately.

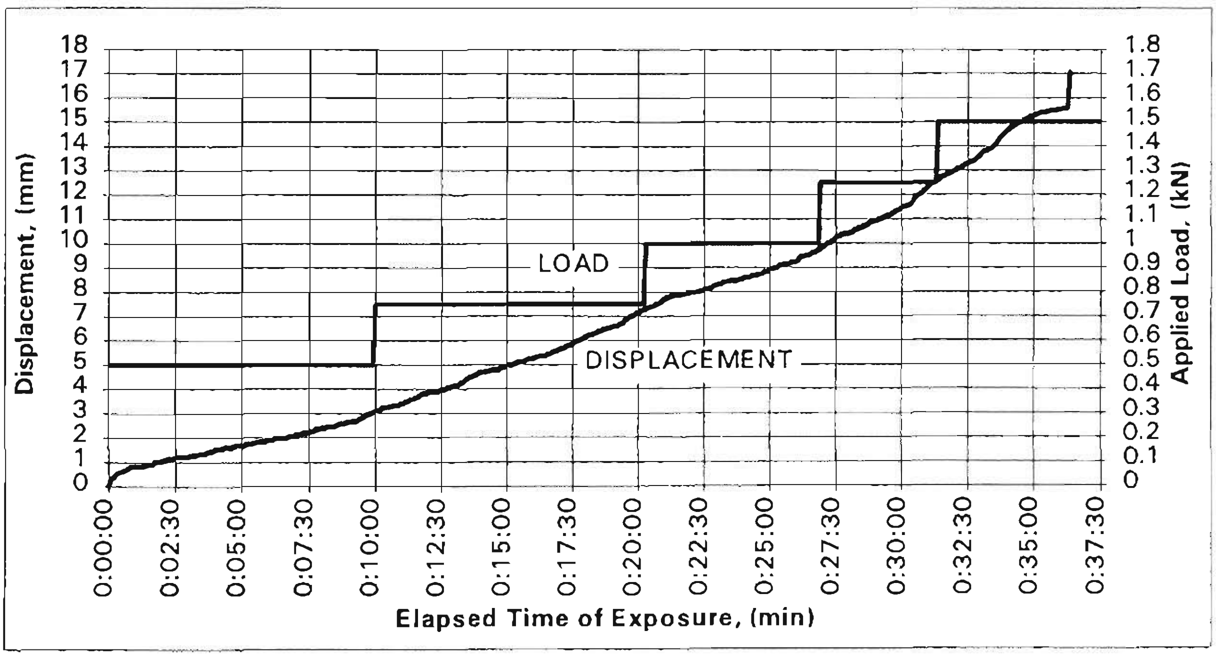


Figure 5.20 Creep Effects for Non-Welded Connection Tested at 800 °C

As noted previously, the non-welded clips eventually failed by separation from the flanges. The same failure mode was noted under both ambient and elevated temperature conditions, (see Figure 5.21 and Figure 5.22). A small amount of rotation of the bolt heads was noted.

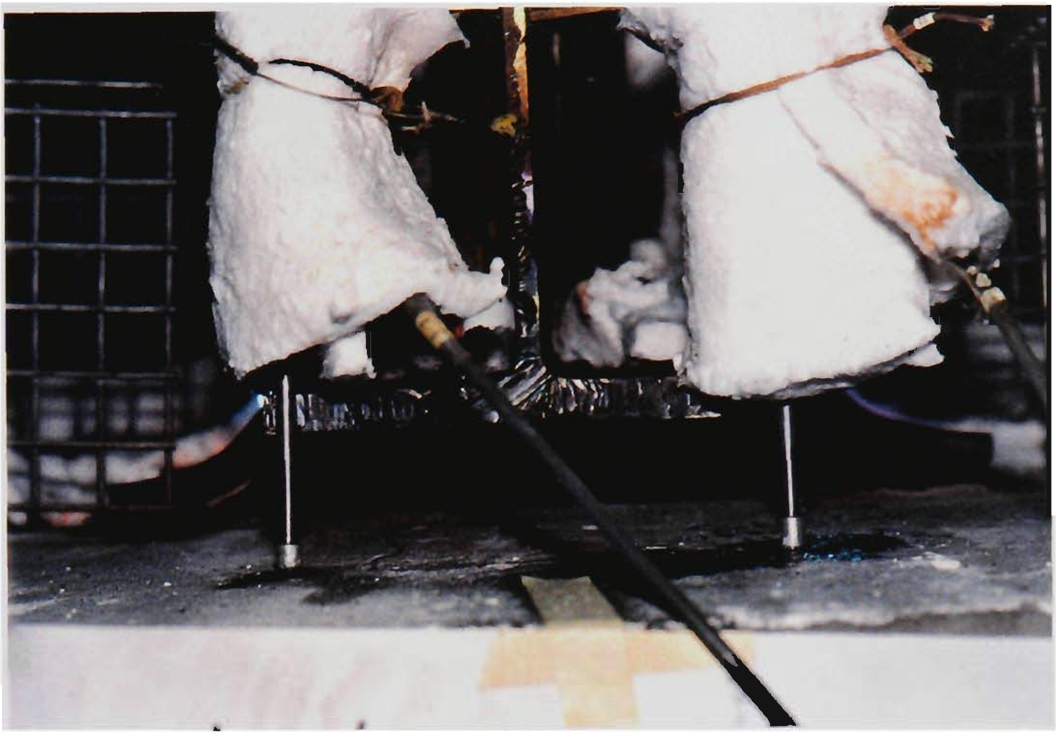


Figure 5.21 Deformation of NW-600 Clip Plate Test

Figure 5.22 Final Plate Deformation of NW-600 Test

5.6.4 Elevated Test / Welded Connection

As was the case for the non-welded tests, the connection specimens were heated to both 600 °C and 800 °C (see Figure 5.8). Again, the clip temperatures were maintained to ±30 °C.

As for the non-welded connections tested at elevated temperatures, the load was increased in 0.5 kN increments. As would be expected, the welded connections, under elevated temperature conditions offered significantly greater stiffness and strength than the non-welded connections.

The displacement-time and applied load-time plots for tests 7 and 8 are given in Figures 5.24 and 5.25, for clip plate temperatures of 600 °C and 800 °C, respectively. As observed in Section 5.6.2, the welded connections did not fail under ambient temperature conditions by bolt failure, weld failure or clip failure, but rather by pulling out of the ferrule from the concrete; ie. anchorage failure.

In contrast to the ambient temperature failure mode, the connections at elevated temperature failed due to the stretching and bending of the bolt shank (see Figure 5.23). In the case of the welded tests, the bolts were subject to much higher loads than the non-welded tests. The displacements for both welded tests (W600 and W800), at failure due to a combination of bolt and clip deformation, were 30 - 40 mm.

It is important to note, that in these tests, the bolt head was heated to the same temperature as the clip plate.

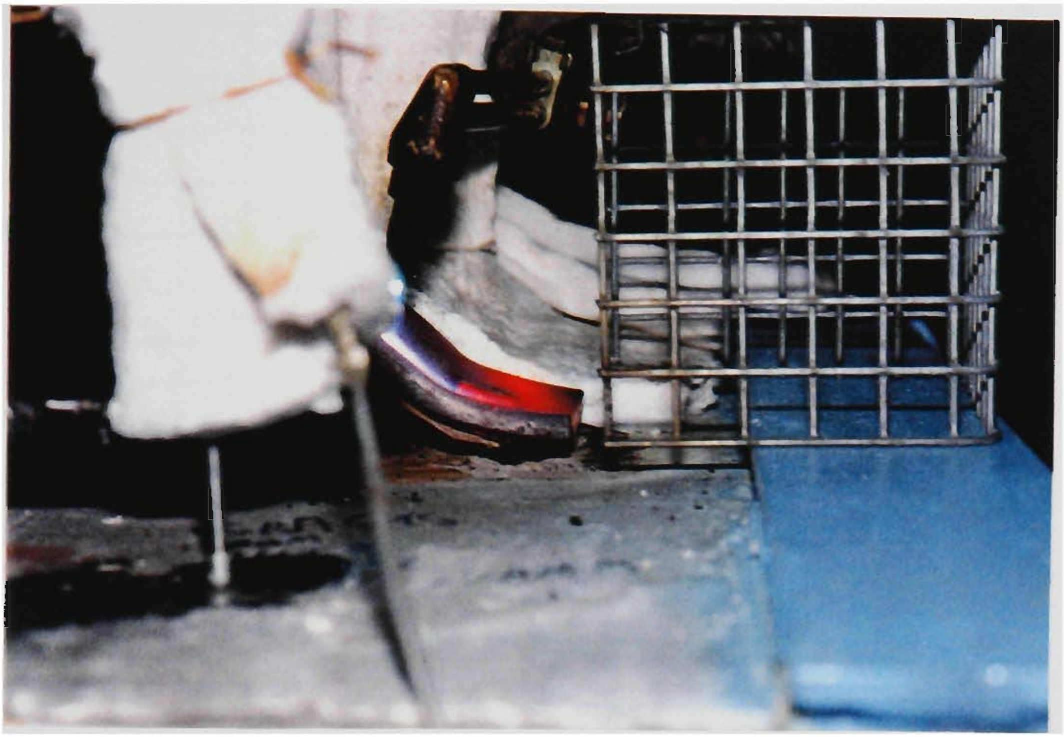


Figure 5.23 Plate and Bolt Deformation of W-600 Test

In a real fire, it is unlikely that the bolt will be at the same temperature due to its relatively low exposed surface area-to-mass ratio and its ability to transfer heat to the concrete slab. Therefore these tests are a conservative representation of what is likely to happen in practice.

The results presented in Figures 5.24 and 5.25 are now considered. For test W600, it can be seen that bolt failure would have occurred with a load level of 5 kN, (5% of the nominal tensile bolt strength), after approximately 4 hours and 30 minutes. A load level of 10 kN; (approximately 10% of the nominal tensile bolt strength) after approximately 3 hours and 30 minutes. Alternatively, if the applied load level was 20% of the nominal tensile bolt strength then the limiting bolt deformations would be achieved in approximately 1 hour and 10 minutes.

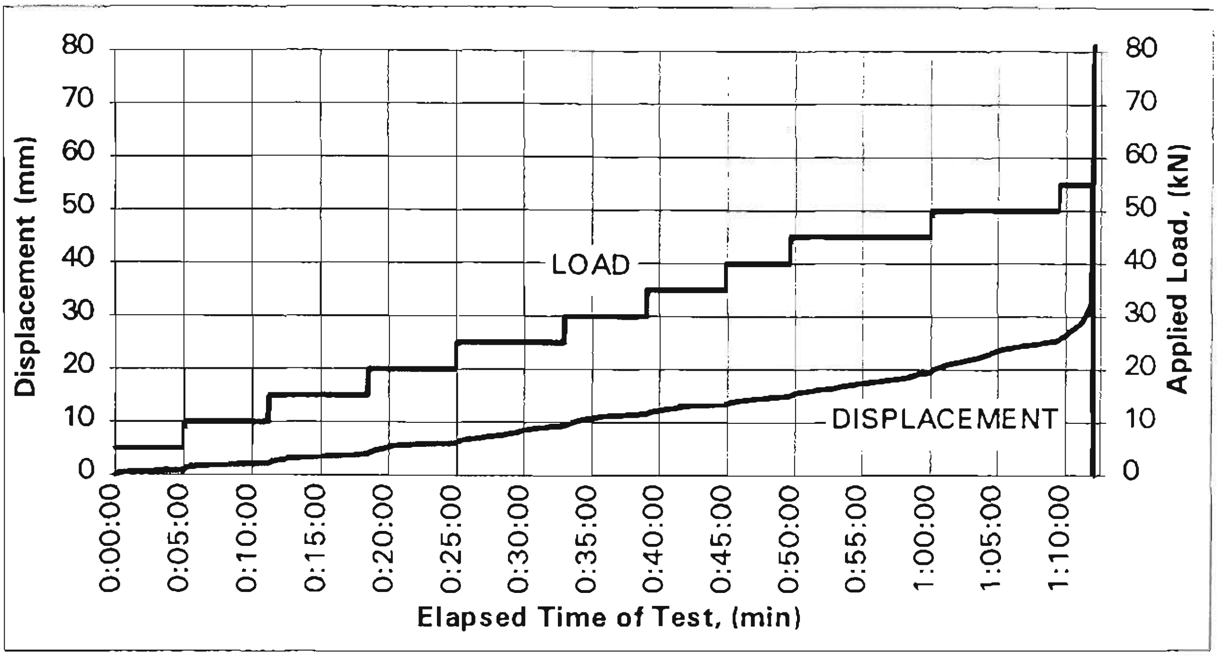


Figure 5.24 Creep Effects for Welded Connection Tested at 600 °C

Some slight deformation of the bolt hole through the plate was noticed - see Figure 5.26. As the clip was bent under the effects of both load and temperature, the bolt was subject to prying and bending (see Figure 5.27). Failure eventually occurred when the bolt suddenly fractured through the thread thus causing the beam section to come away from the slab. Failure was sudden and occurred on one side of the column section with the opposite side failing immediately after, due to the immediate transfer of load. Throughout the test the welds remained intact.

The results for the 800 °C test (W800) are now considered in detail (see Figure 5.25).

The maximum load experienced by the connection at 800 °C was approximately 13 kN with the maximum deflection at this load being 37 mm.

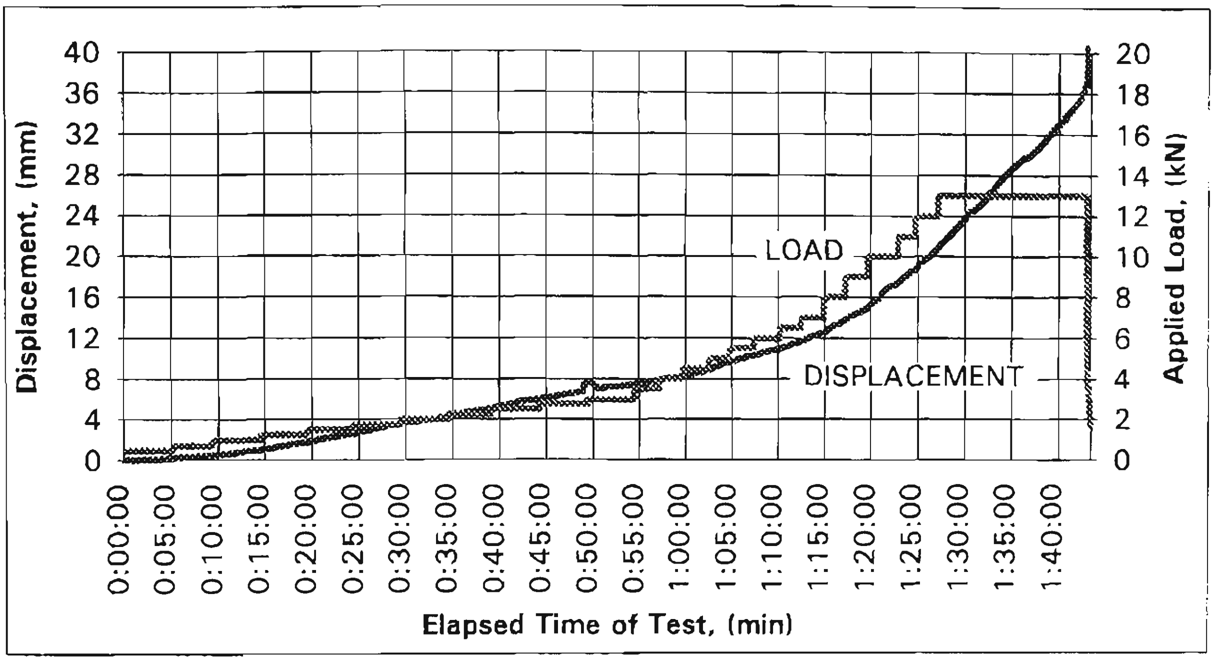


Figure 5.25 Creep Effects for Welded Connection Tested at 800 °C

Again, failure was by stretching and bending of the bolt. Bolt failure occurred when the plate separated from the slab, at the flange edge, by approximately 35 mm and occurred at a lower load level than achieved for the W600 test. Compared with the W600 test, it can be seen that the rate of displacement increased substantially for the same load level.

Figure 5.26 Failure of W-800 Test Showing Bolt Yielding

Based on a limiting relative displacement of 35 mm, it has been determined from Figure 5.25 that failure would occur for a load level of 5 kN, (5% of the nominal ambient tensile bolt capacity), after approximately 2 hours and 30 minutes and for a load level of 10 kN, (10% of nominal ambient tensile bolt capacity), after approximately 90 minutes. If a load level of approximately 20 kN, (20% of the nominal tensile bolt capacity) was chosen, then failure would be almost immediate.

The clip plate gradually deformed under the effects from both temperature and load to the point where the bolts failed. As was the case for the 600°C welded specimen, the bolts elongated and failed quite suddenly through the bolt thread (see Figure 5.26 and Figure 5.27).

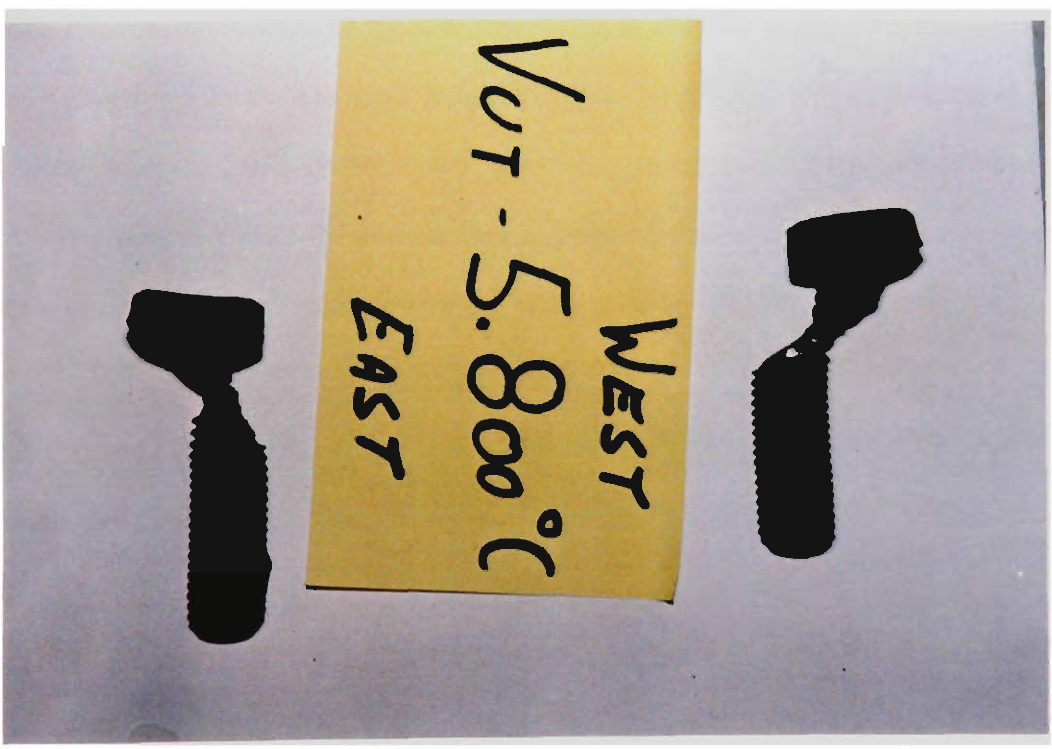


Figure 5.27 Bolt Deformation of W-800 Test

5.7 Conclusions

This chapter reports the results of ambient and elevated temperature tests on two types of connections - non-welded clip plate connections and welded clip plate connections. These connections are the most common in practice.

The following conclusions may be drawn from the tests reported in this chapter:

- Models for ambient temperature strength were proposed for estimating the likely strength of the connections. These were found to correlate adequately with the test results.

- Under both ambient and elevated temperature conditions, the failure mode associated with the non-welded clip was separation of clip and flange due to opening of the clip. For both ambient and elevated temperatures this was found to occur when the clip displacement reached approximately 16 mm. This displacement is unlikely to be sufficient to absorb the relative displacement between horizontal panels and supporting frame, based on the likely relative displacements calculated for such situations in chapter 4.

- Under ambient conditions, the welded clip connection failed by the ferrule being pulled out of the concrete slab.

- Under elevated temperature conditions, the welded clip plate connection failed due to excessive bolt deformation. For both 600 °C and 800 °C, a limiting combined plate plus bolt displacement of approximately 35 mm was obtained. This will offer sufficient ductility for horizontal panels. The use of a larger diameter bolt would improve the elevated temperature strength of the connection, and this may be important for the support of vertical panels where higher forces must be resisted.

- Welded connections are able to resist much greater load than the non-welded connections. This is true for both ambient and elevated temperature conditions.

- Time dependent deformation increased with both temperature and load level. This was the case for both types of connection.

- Approximate estimates of the time to failure at 600°C and 800 °C were made for various load levels for both types of connection. The findings are as follows:

Table 5.4 Estimated Time to Failure of the Elevated Connection Tests

	Estimated Time to Failure; (Hours, Minutes)		
	Percentage of Nominal Ambient Tensile Bolt Capacity; (%)		
Test Type	*5*	*10*	*20*
NW600	14 hrs	4 hrs, 30 min	2 hrs, 30 min
NW800	Immediate	Immediate	Immediate
W600	4 hrs, 30 min	3 hrs, 30 min	1 hr, 10 min
W800	2 hrs, 30 min	1 hr, 30 min	Immediate

- The welded connection is much more resistant to time-dependent deformations, at a given load level and temperature, then the non-welded connection.

As far as the support of vertical panels is concerned, (Figure 5.1(a) and (c)), it is clear that neither the welded or non-welded connection are capable of achieving, in themselves, the limiting relative displacements between wall panel and column as discussed in Chapter 4. The maximum 'ductility' offered by the welded connection, while still maintaining load, was about 35 mm. This is an order of magnitude less than that likely to be required for vertical panels, but is likely to be satisfactory for the support of horizontal panels.

Welded connections should only be used for ***vertical*** panels where they are used in combination with another member which can provide for the relative displacement between panel and frame - eg. an eaves tie member.

CHAPTER 6 CONCLUSIONS

This thesis has considered aspects of the behaviour of single storey industrial buildings consisting of steel portal frames and external concrete wall cladding. Such buildings frequently have vertical and horizontal concrete panels and these are connected to the steel supporting frame using a variety of connection systems. As shown in chapter 2, most of the current connection systems utilise a steel clip plate which is connected into the concrete panel and then to the supporting member or frame. The behaviour of this particular connection is the subject of chapter 5.

Concern regarding the behaviour of such buildings in fire has resulted in clause C1.11 being incorporated in the BCA. For buildings with external concrete walls, this clause requires buildings to be designed such that, in the event of a fire, the wall panels will not collapse outwards. Various design philosophies have been developed to achieve this outcome and these were reviewed in chapter 2. It was concluded that it is important that the wall panels remain firmly attached to the frame throughout the fire.

In chapter 3, the development of a numerical model to study the behaviour of concrete wall panels under fire conditions was developed. This model is based on several simplifying assumptions including that of single curvature bending and that the elevated temperature behaviour of reinforcing steel and concrete could be described by 'simplified' stress-strain relationships.

The model was found to give similar answers to those given by the more sophisticated model of O'Meagher et al. (1991).

The model was used in chapter 4 to study the behaviour of isolated cantilever walls and walls that are restrained at the top. It was found that typical cantilevered walls with representative areas of reinforcement could not achieve a fire-resistance of more than 30 minutes when exposed to a standard fire over their entire height (6m case). However, if the isolated walls were exposed to fire only over part of their height, the fire-resistance was increased by as much as a factor of three. Ultimately, isolated walls will collapse outwards.

For walls restrained at the top, it was shown that due to prying against the column flange, vertical panels will behave as restrained cantilevers - irrespective of their base fixity details. Therefore as the fire progresses inside a building, vertical panels will bow outwards - away from the column flange. O'Meagher et al. (1994) have proposed formulae to give the maximum relative displacement between a panel and its' supporting structure prior to collapse of the frame. Using the model developed in chapter 3, the behaviour of typical restrained vertical panels was studied. It was found, assuming full height fire exposure, that the relative displacements obtained from O'Meagher's formulae would be achieved after only 10 minutes of standard fire exposure. This was noted as being significantly less than the 20 - 30 minutes of similar fire exposure required to cause gross deformation. Nevertheless, it was concluded that the displacements obtained from O'Meagher's formulae are likely to be of the correct order of magnitude - especially when it is considered that the simplifying wall model assumption of single curvature bending has been made and that it is unlikely that a wall will be subject to uniform heating up its' height. It is recommended that some analysis be undertaken to determine the significance of the assumption regarding single curvature bending.

Simplified expressions were given for the capacity of a connection if located at the top of a vertical panel.

The detailed behaviour of horizontal panels attached to steel frames was also considered. Initially, the clip connections may be subject to high forces due to prying of the panels against the column flange. As the columns on one side of the frame move outwards, additional P-δ forces are developed. This allows the prying effects to be relieved and the forces to be substantially reduced. Based on statics and an assumed geometry, an expression for the magnitude of the final force on horizontal connections was derived.

As noted previously, chapter 5 reports an extensive experimental program which looked at the behaviour of the most common form of connection - the clip plate - under both ambient and elevated temperature conditions. Two types of connections were considered; non-welded and welded.

The non-welded clip failed by the clip opening up due to plastic deformation. This was true for both ambient and elevated temperature conditions. The welded clip offered substantially greater capacity at both ambient and elevated temperature conditions.

These tests illustrated the importance of creep and showed that time-dependent deformation increases with increased temperature and increased load level. For design purposes, it is necessary to keep the load level of the connection down to a sufficiently low level to minimise creep deformations. Guidance can be obtained from the results of these tests and it would appear that, at 800 °C, the load ratio for the welded clip should be kept at around 0.05. This conclusion is likely to also apply to other types of connections where (unlike the non-welded clip) the connection cannot disengage as it deforms. At 800 °C, it was found that the non-welded clip opened up and failed with almost no load applied to it.

Neither the non-welded or welded clip plate connections were found to be able to achieve the relative displacements between vertical panel and supporting frame as recommended by O'Meagher et al. (1994). This is an important observation as this form of connection is commonly used to connect vertical panels directly to the column flange. If this practise continues, it is likely that such panels will collapse outwards away from the supporting frame.

Such connections can be used for vertical panels provided they are combined with a support member (such as an eaves tie) which is capable of allowing for relative movement between panels and frame.

Alternatively, the load-deformation information on welded clip plate connections could be used to develop a specific connection which has sufficient strength and is capable of plastically deforming to an appropriate level of relative displacement.

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APPENDIX A CASE STUDIES

A1.0 Introduction

This appendix presents a number of case studies of steel framed buildings that have been subjected to significant fires in recent years. This is not an exhaustive study covering all fires, that have occurred in such buildings, but is aimed at illustrating a number of behavioural aspects.

A2.0 Case Studies

Case 1 TRANSPORT MANAGEMENT FACTORY

Mayne Nickless, (Melbourne 1992)

Construction - Construction consisted of steel portal frames with attached tilt-up vertical concrete wall panels. The concrete panels were approximately 150mm thick and were 'pin-jointed' at the bottom of the structure, and pursuant to AS3600, deemed to have an FRL equivalent to 240/240/240. The roof structure consisted of metal decking supported by steel roof purlins spanning between the rafters. The floor area of the building was approximately 4200m^2 and was compartmented by a fire wall into two sections of approximately 2100m^2 in area.

The concrete panels were connected directly to the outer flanges of column by welded clips and dynabolts (see Figure 2.4). Similar connections were provided between the eaves tie member and the panels except that in this case, the clips were not welded to the eaves tie (see Figure 2.6).

Regulatory Requirements - Pursuant to the BCA 1990, this building was deemed to comply with the regulations and had a rise in storeys of one (1) and was of Type C construction. However due to each compartment having an area of 2100m^2, Type B construction was required in accordance with Table C2.2 of Specification C1.1.

Fire Load - The contents of the building was diverse in type and quantity and included: calcium hypochlorite (60-70 tonnes), hydrochloric acid (4 tonnes), cooking oil (100 tonnes), white goods, metal ware, general consumer goods.

Fire Initiation and Spread - Fire initiated in one compartment and spread into the adjacent compartment by means of a fire door that had been left open. There was no fire spread to adjacent buildings primarily due to adequate separation provided by the external walls of the fire affected building and adjacent buildings as well as the actions of the attending fire fighting crews. The fire burned for more than six hours.

Building Behaviour - Throughout the course of the fire, several tilt-up concrete wall panels moved inwards and outwards, (mostly inwards), with the majority of the remainder of panels leaning precariously in both directions. Observation of the building after the fire indicated that the dynabolts connecting the panels to the column clips had been pulled out of the concrete, and that the clips connecting the panels to the eaves tie member has either grossly deformed or rotated, permitting separation of panel and supporting structure.

Figure A.1 Case 1 - Mayne Nickless Storage Warehouse

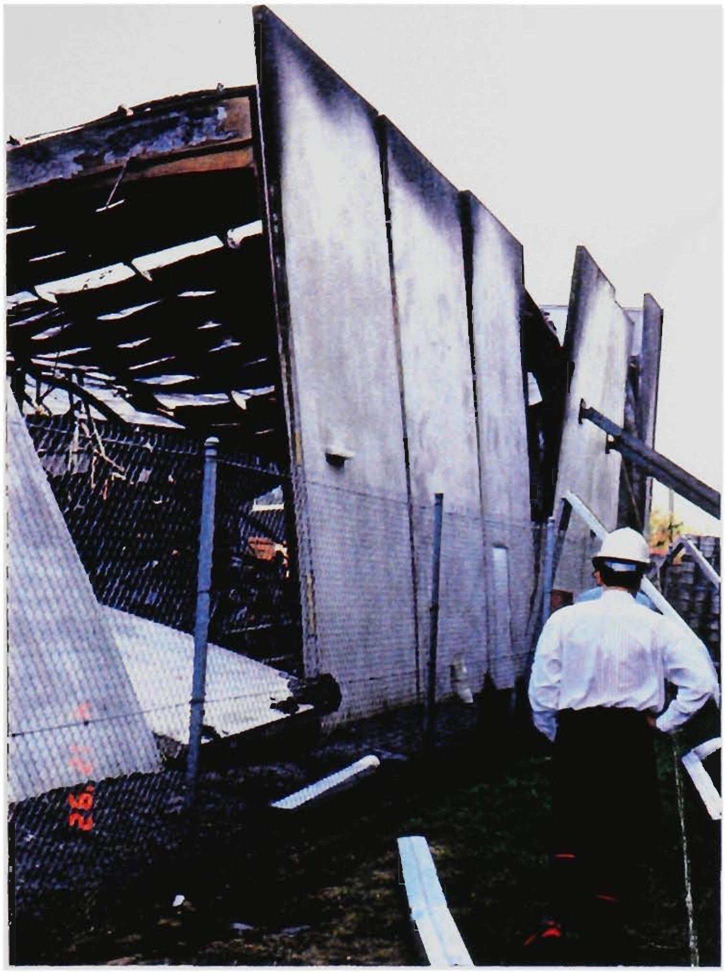


Figure A.2 Case 1 - Mayne Nickless Storage Warehouse

Figure A.3 Case 1 - Mayne Nickless Storage Warehouse (Melbourne, 1992)

Figure A.4 Case 1 - Mayne Nickless Storage Warehouse (Melbourne, 1992)

Figure A.5 Case 1 - Mayne Nickless Storage Warehouse (Melbourne, 1992)

Case 2 *PAINT STORAGE WAREHOUSE*

Colortron Pty Ltd, (Melbourne 1987)

Construction - Construction consisted of a steel portal frame with attached precast horizontal concrete wall panels. The concrete wall panels were approximately 150 mm thick and were connected to the steel portal frame via a series of clip connections (see Figure 2.4). The roof structure consisted of steel sheeting supported by steel purlins. The floor area of the building was approximately 1670 m^2 which consisted of factory and office space.

Regulatory Requirements - Pursuant to the now replaced VBR's (Victorian Building Regulations), this building was classed as a two storey building and the minimum type of construction required for this building was Type 5. The building was of mixed class of occupancy - Class V (office) and Class VIIIB (warehouse).

Fire Load - The exact quantity and type of goods stored within the building was not able to be established with accuracy but it is known that the goods included a substantial quantity of paints and industrial strength solvents.

Fire Initiation and Spread - There was no fire spread to adjacent buildings primarily due to adequate separation provided by the fire-resistant walls and the actions of the attending fire fighting crews.

Building Behaviour - As can be seen from the following photographs showing the post-fire results, the steel roofing membrane collapsed under it's own self-weight through loss of strength at elevated temperatures. This behaviour resulted in the supporting columns being pulled inwards. The clip connections between the precast concrete wall panels and the supporting steel frame and failed throughout the course of the fire. These clip connections were not welded to the column flanges.

Figure A.6 Case 2 - Colortron Paint Storage Warehouse (Melbourne, 1987)

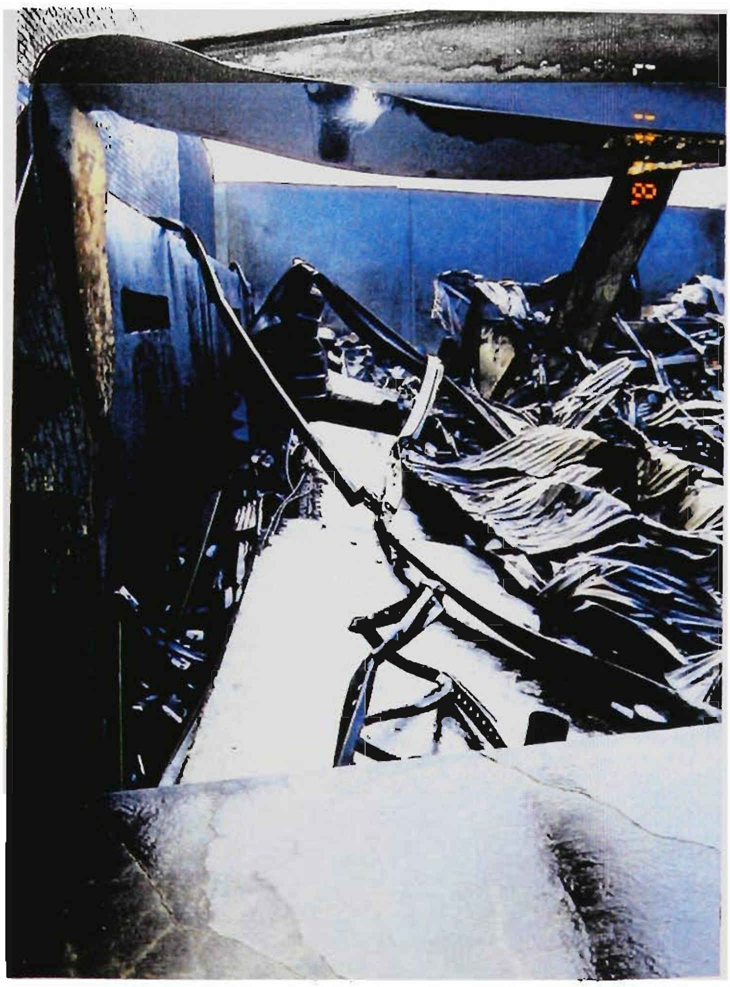


Figure A.7 Case 2 - Colortron Paint Storage Warehouse (Melbourne, 1987)

Figure A.8 Case 2 - Colortron Paint Storage Warehouse (Melbourne, 1987)

Case 3 *POOL CHEMICALS WAREHOUSE*

(Sydney 1988)

Construction - The building consisted of unprotected steel portal frames and vertical concrete wall panels connected to the steel frames. The wall panels were 6m high, 9m long and 140 mm thick.

Fire Load - The fire load was very high consisting of pool chemicals with about 70 tonnes of chlorine stacked in one area of the building.

Fire Initiation and Spread - The closest building was located about 3m away on one side. No spread of fire to any of the adjacent buildings occurred.

Building Behaviour - The steel frames deformed inwards with the rafters sagging downwards and the columns correspondingly moved inwards. Most of the concrete panels moved inwards with the steel frame but two panels which became detached from the steel frame due to inadequate connection and collapsed outwards. It can be seen from the attached photographs that the failure of these connections was due to the development of prying forces at the connection and its inadequate strength and ductility.

Figure A.9 Case 3 - Pool Chemicals Warehouse (Sydney, 1988)

Figure A.10 Case 3 - Pool Chemicals Warehouse (Sydney, 1988)

Figure A.11 Case 3 - Pool Chemicals Warehouse (Sydney, 1988)

Case 4 TOY FACTORY AND WAREHOUSE

(Melbourne 1989)

NOTE: *Although this building is not a typical portal frame structure comprising of precast / tilt-up concrete panels used as cladding it is seen as an important example showing the behaviour of such buildings in fire.*

Construction - The factory/warehouse construction consisted of steel portal frames with fire protected columns with masonry walls and an unprotected roof. The masonry walls were supported by the portal frames. The building housed some offices at the front of the factory/warehouse construction.

Fire Load - The fire load was very high consisting of timber, glue, cardboard and packaging. The fire was very severe and burnt for more than six hours.

Fire Initiation and Spread - An adjacent building was located about 6m away but no fire spread occurred.

Building Behaviour - Movement of the steel frames and explosions within the building resulted in the early collapse of the brick wall along one side of the building. The brick encasements around the portal frame columns also became dislodged. The rafters deformed downwards until they were in contact with the floor with the columns being pulled inwards.

Figure A.12 Case 4 - Toy Factory and Warehouse (Melbourne, 1989)

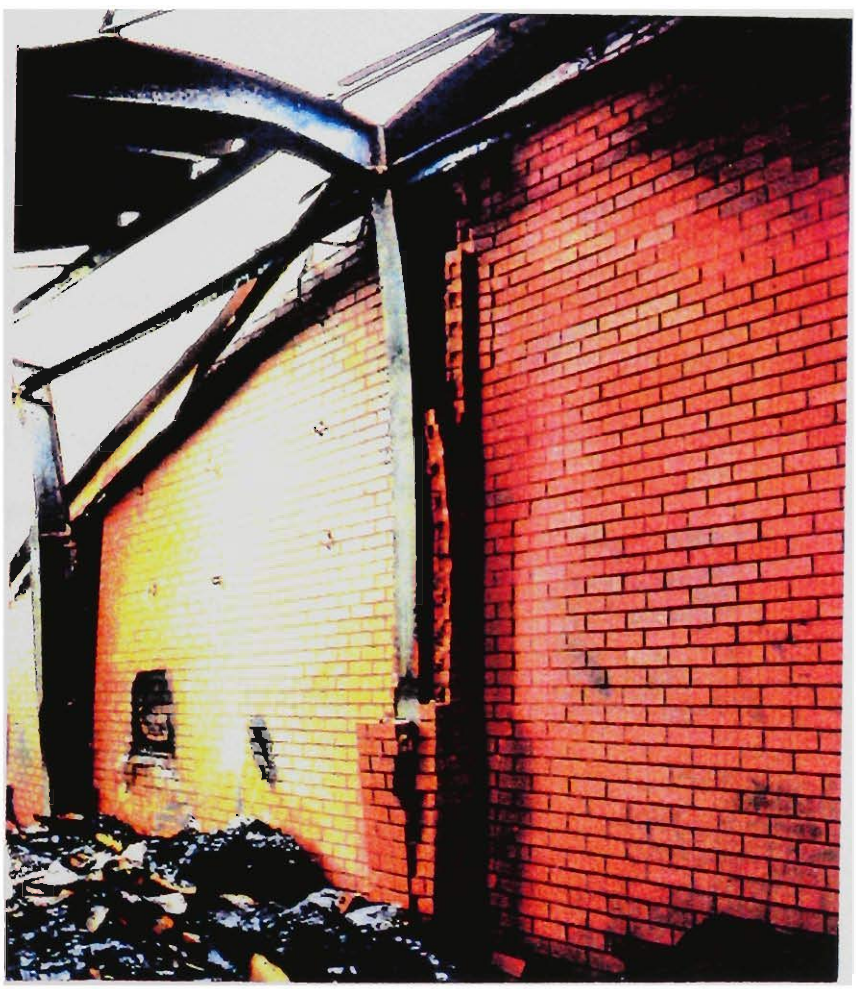


Figure A.13 Case 4 - Toy Factory and Warehouse (Melbourne, 1989)

Case 5 CARPET STORAGE WAREHOUSE

Carpet Call, (Brisbane 1992)

Construction - Construction consisted of steel portal frames with non-loadbearing reinforced concrete tilt-up wall panels. The concrete wall panels were approximately 8 m in height and 3 m in width. The only fixings of the panels to the steel columns were four sets of two metal clips measuring 125 mm × 75 mm × 8 mm which held the panels in position against the columns. The wall panels were simply supported at the base.

Fire Load - The fire load was not exactly known but consisted of vast quantities of carpet and various other indoor furnishings.

Fire Initiation and Spread - The fire did not spread to any adjacent buildings even though panels did fall outwards.

Building Behaviour - After a period of approximately 30 minutes after fire initiation, the tilt-up panels attached to the portal frames collapsed outwards. Of the panels that did not collapse outwards, the behaviour was either of badly distorted panels with an outwards camber of approximately 150 mm longitudinally as well as a 50 mm distortion taking place across the breadth of the panels at the top. The west wall panels fell out and on to an adjacent toilet block (unoccupied at the time) whilst the south wall panels failed and lent against an adjacent building. The portal frame rafters collapsed inwards.

Figure A.14 Case 5 - Carpet Call Warehouse

Figure A.15 Case 5 - Carpet Call Warehouse

Figure A.16 Case 5 - Carpet Call Warehouse

Figure A.17 Case 5 - Carpet Call Warehouse

Figure A.18 Case 5 - Carpet Call Warehouse

B3.0 Thermal Properties

In addition to the stress-strain relationships it is also necessary to take account of the thermal expansion of both concrete and reinforcing steel. Models as given in Eurocode 4 are described below.

B3.1 Steel

The values of thermal elongation for all structural and reinforcing steels over varying temperatures is given by the following:

$$\left(\frac{\Delta l}{l}\right)_s = -2.416\times10^{-4} + 2\times10^{-5}.(\theta_s) + 0.4\times10^{-8}.(\theta_s)^2 \quad \text{for } 20^\circ C \le \theta_s \le 750^\circ C$$

$$= 11\times10^{-3} \quad \text{for } 750^\circ C < \theta_s \le 860^\circ C$$

$$= -6.2\times10^{-3} + 2\times10^{-5}.(\theta_s) \quad \text{for } 860^\circ C < \theta_s \le 1200^\circ C$$

where:

l_s	=	length at room temperature, (m)
$(\Delta l)_s$	=	temperature included elongation, (m)
θ_s	=	steel temperature, (°C)

B3.2 Concrete

The values of thermal elongation for concrete over varying temperatures is given by the following:

$$\left(\frac{\Delta l}{l}\right)_c = -1.8\times10^{-4} + 9\times10^{-6}.(\theta_c) + 2.3\times10^{-11}.(\theta_c)^3 \quad \text{for } 20^\circ C \le \theta_c \le 700^\circ C$$

$$= 14\times10^{-3} \quad \text{for } 700^\circ C \le \theta_c \le 1200^\circ C$$

where:

l_c	=	length at room temperature, (m)
$(\Delta l)_c$	=	temperature induced elongation, (m)
θ_c	=	concrete temperature, (°C)

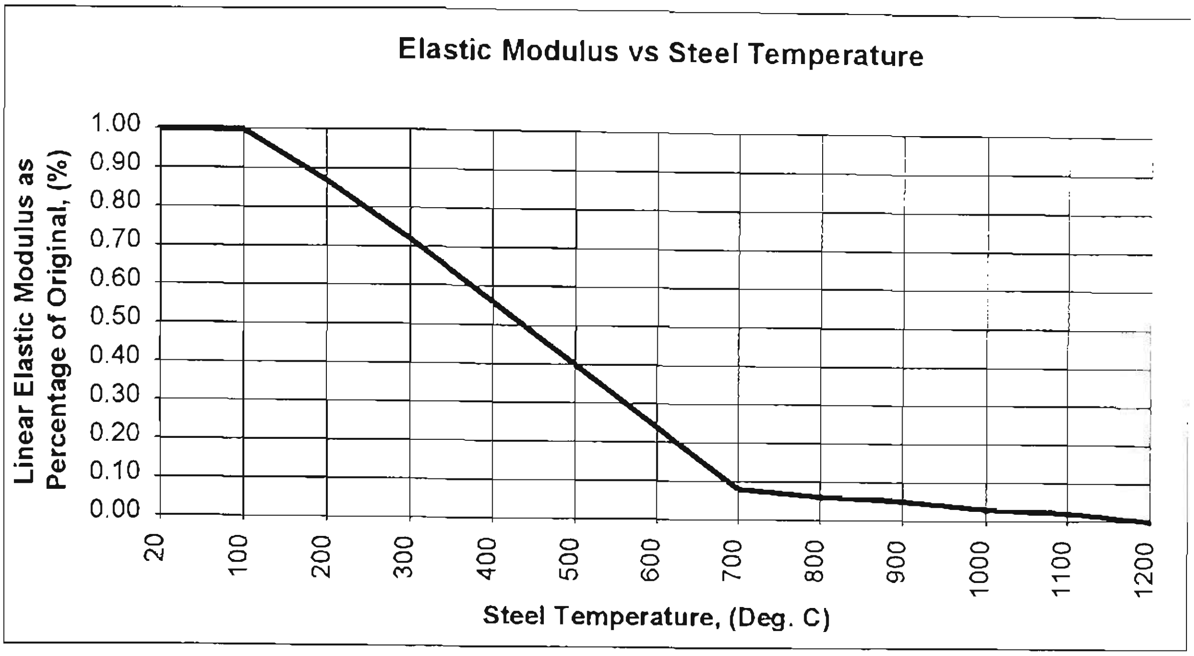


Figure B.1: Elastic Modulus versus Temperature - (Steel)

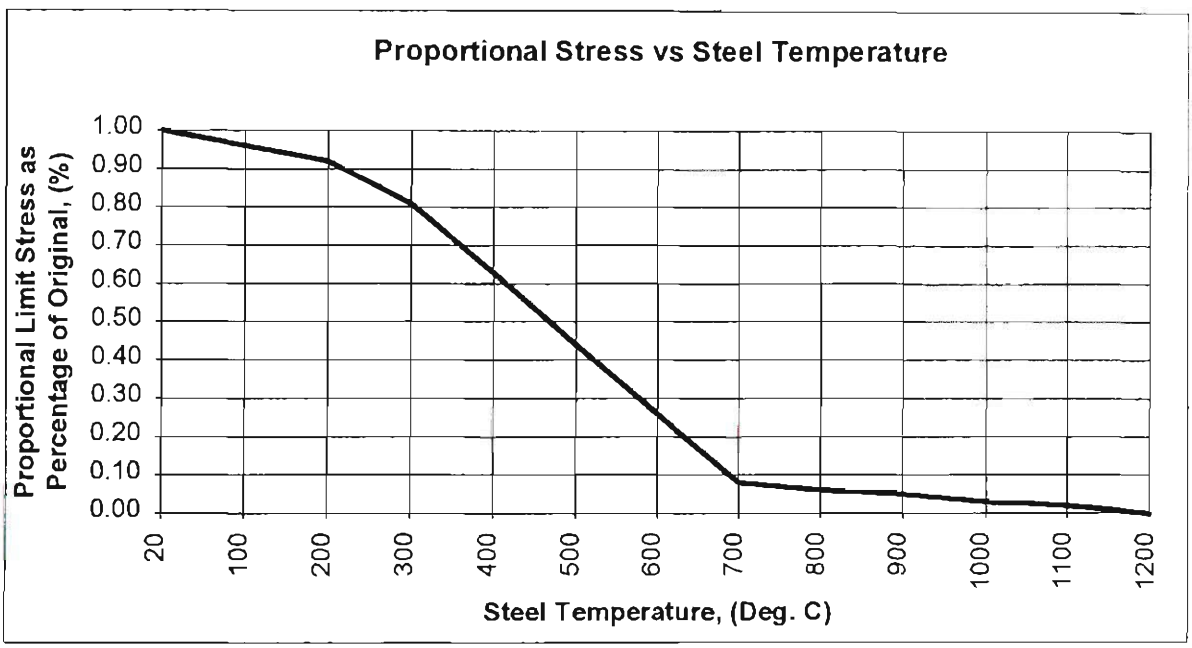


Figure B.2: Proportional Limit Stress versus Temperature - (Steel)

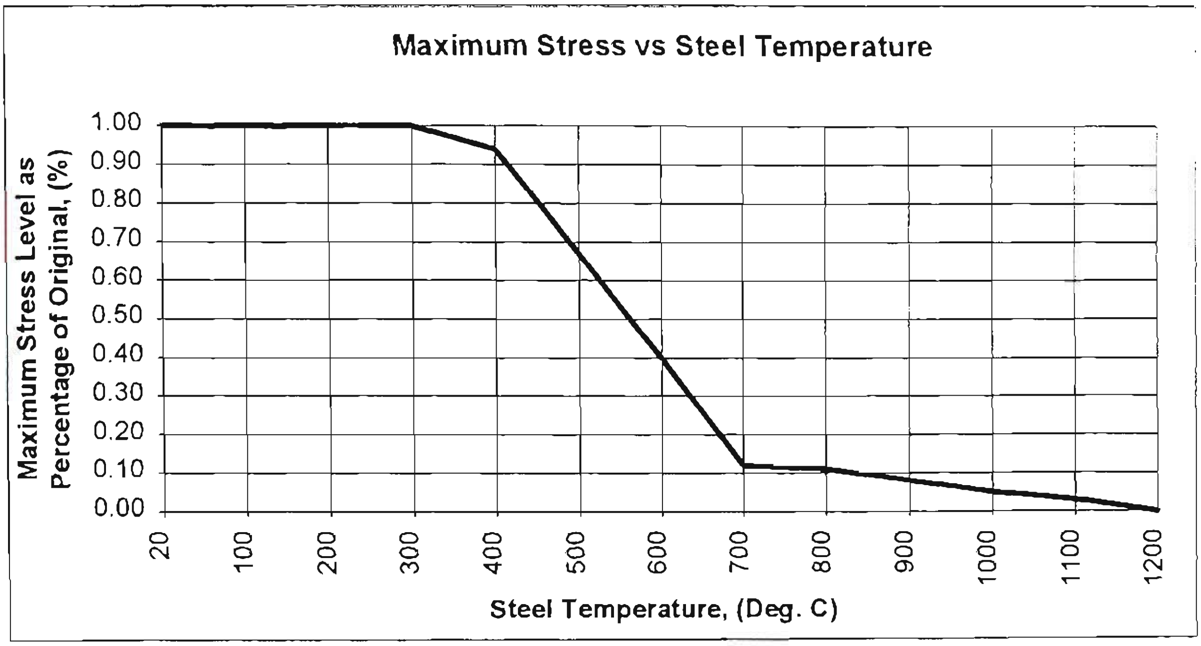


Figure B.3: Maximum Stress Level versus Temperature - (Steel)

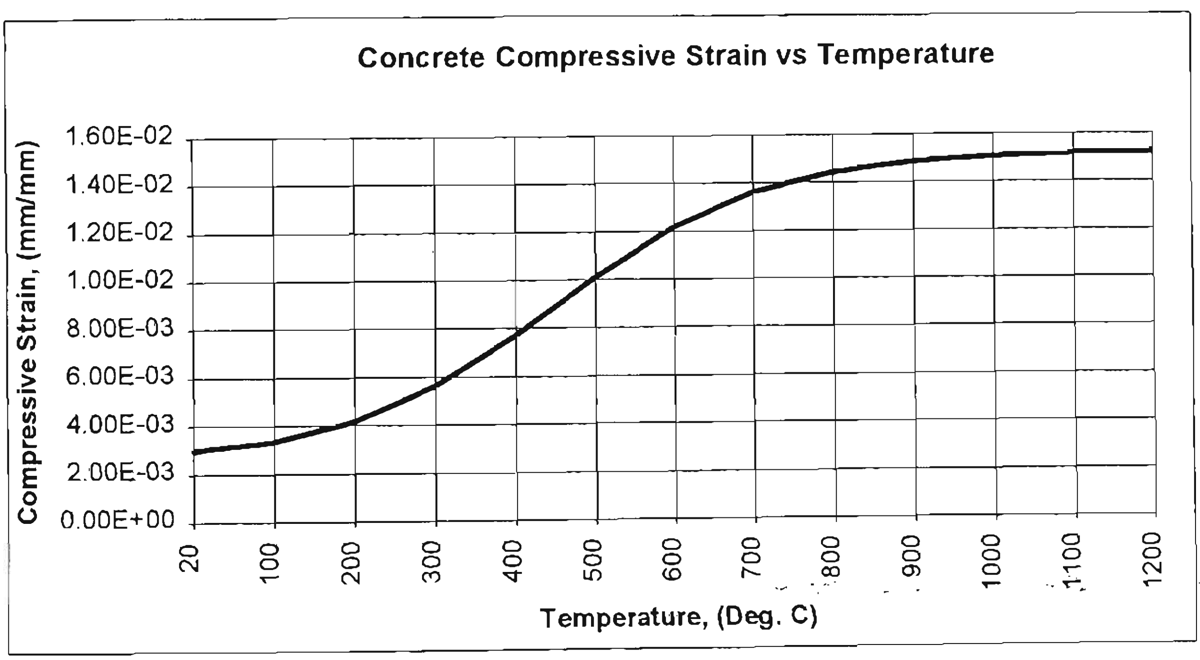


Figure B.4: Compressive Strain versus Temperature - (Concrete)

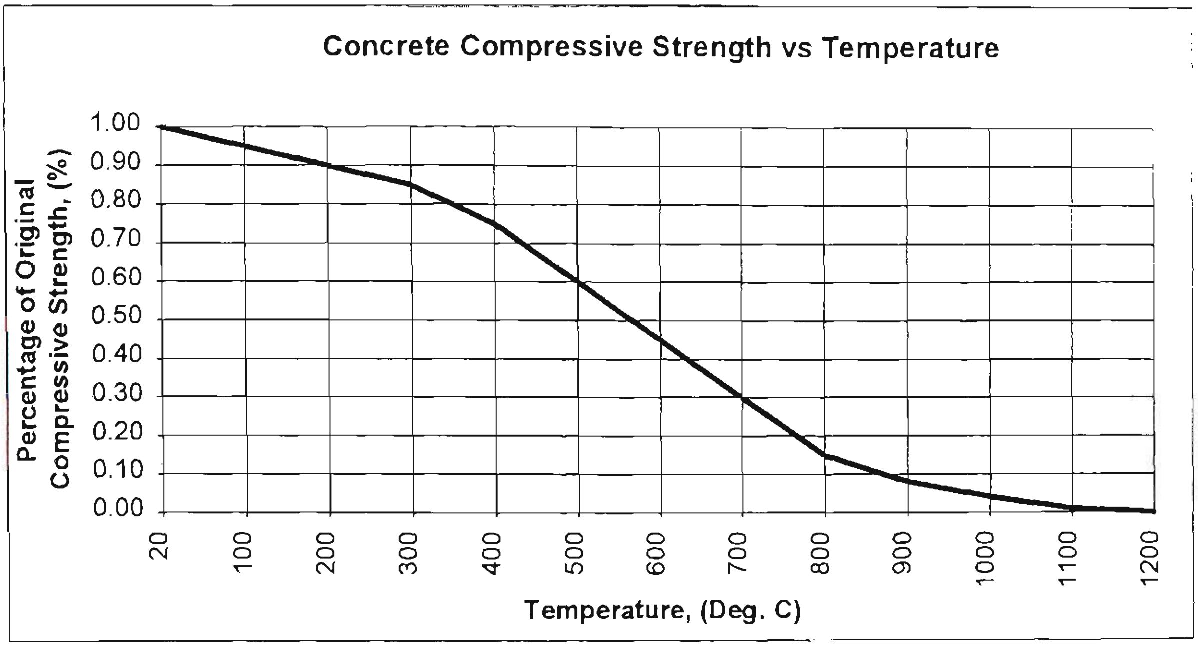


Figure B.5: Compressive Strength versus Temperature - (Concrete)

APPENDIX C WALL MODEL PROGRAM LISTING

C1.0 Introduction

Provided in this Appendix is the entire program listing of the wall model described in Chapter 3.

The program listing was written in Q-Basic format.

C2.0 Wall Model Program Listing

```
REM ************************* WALL MODEL PROGRAM LISTING ***************************
REM *********** DEVELOPED FOR MASTERS STUDY INTO BEHAVIOUR OF ************
REM ****** CANTILEVERED CONCRETE WALLS AT ELEVATED TEMPERATURES *****
REM ******************** DATE OF LAST EDIT - 2 SEPTEMBER 1994 **********************

DEFDBL A-Z
DIM LATMOM(26), SWM(26), D(26), DELTA(30), CONCSTR(10), FC(10), EC(10), T(10), CURV(26), PPSI(26),
THETA(26), DISP(26), LENGTH(9), DIST(10), ALPHAC(10), ALPHAS(10), THSTRAINC(10), THSTRAINS(10), EM(10),
ETOT1(10), ETOT2(10), ETOT3(10), STSTRAIN1(10), STSTRAIN2(10), STSTRAIN3(10), FORCE1(10), FORCE2(10),
FORCE3(10), MOM1(10), MOM2(10), MOM3(10), ET(10), STMOD(10), PRSTRESS(10), MSTRESS(10), PRSTRAIN(10),
AVDELTA(26)
CLS

REM ************ USER INPUT ************
INPUT "IS THIS THE START OF A NEW ANALYSIS"; RUN$
INPUT "WHAT IS THE FIRE EXPOSURE - 2.5, 5, 7.5, 10, 15, 20, 25, 30, 60, OR 90 MIN"; ISO
INPUT "WHAT IS THE REINFORCEMENT RATIO"; PW
INPUT "WHAT IS THE WALL HEIGHT - 6m or 8m"; H
INPUT "HOW MANY ELEMENTS UP THE WALL"; G
INPUT "WHAT ELEMENT HAS THE STEEL REINFORCEMENT,IE. X"; X
INPUT "DO YOU WANT SELF WEIGHT EFFECTS - (Y/N)"; SW$
INPUT "WHAT IS THE AXIAL FORCE - EG. 0kN, 5kN, 10kN"; N
INPUT "WHAT IS THE LATERAL LOAD - 0EG. kN, 0.25kN, 0.5kN, 0.75kN, 1.0kN"; P
INPUT "HOW MANY OF THE ABOVE ELEMENTS DOES THE ISO FIRE ACT"; K

CLS

COUNTER = 0
W = 1
L = (H / G) * 1000
AREA = 15 * L
56 IF SW$ = "Y" OR SW$ = "y" THEN 57 ELSE 60
57 SW = .15 * W * (L / 1000) * 25
60 AST = PW * .15 * 1 * 1000000
EO = 200000
FAY = 400
FCO = 32
PPSI(1) = 0
DISP(0) = 0
THETA(0) = 0
SWMTOT = 0
```

```
REM ************ READ TEMPERATURE DISTRIBUTIONS ************
FOR I = 1 TO 10
     READ ISO2.5A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO2.5B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO5A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO5B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO7.5A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO7.5B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO10A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO10B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO15A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO15B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO20A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO20B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO25A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO25B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO30A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO30B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO60A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO60B(I)
NEXT I
FOR I = 1 TO 10
     READ ISO90A(I)
NEXT I
FOR I = 1 TO 10
     READ ISO90B(I)
NEXT I

REM ************ READ CENTROID DISTANCES ************
FOR I = 1 TO 10
READ DIST(I)
NEXT I

REM ************ READ DEFORMED SHAPE ************
90 IF RUN$ = "YES" OR RUN$ = "yes" THEN 91 ELSE 92
```

```
91    OPEN "B:\DELTA1.DAT" FOR INPUT AS #1
      FOR J = 1 TO G + 1
      INPUT #1, DELTA(J)
      IF EOF(1) THEN CLOSE
      NEXT J
      CLOSE #1

GOTO 93

92    OPEN "B:\DELTA.DAT" FOR INPUT AS #2
      FOR J = 1 TO G + 1
      INPUT #2, DELTA(J)
      IF EOF(2) THEN CLOSE
      NEXT J
      CLOSE #2

93 REM ************ CALCULATE LATERAL MOMENT ************
FOR J = 1 TO G
LATMOM(J) = P * (J * L / 1000)
NEXT J

REM ************ INITIATE T1 AND T2 ZONES ************
94 IF ISO = 2.5 THEN 95 ELSE 100
95    FOR I = 1 TO 10
96          LET T1(I) = ISO2.5A(I)
97          LET T2(I) = ISO2.5B(I)
98    NEXT I
99 GOTO 510
100 IF ISO = 5 THEN 110 ELSE 151
110   FOR I = 1 TO 10
120         LET T1(I) = ISO5A(I)
130         LET T2(I) = ISO5B(I)
140   NEXT I
150 GOTO 510
151 IF ISO = 7.5 THEN 152 ELSE 160
152   FOR I = 1 TO 10
153         LET T1(I) = ISO7.5A(I)
154         LET T2(I) = ISO7.5B(I)
155   NEXT I
156 GOTO 510
160 IF ISO = 10 THEN 170 ELSE 220
170   FOR I = 1 TO 10
180         LET T1(I) = ISO10A(I)
190         LET T2(I) = ISO10B(I)
200   NEXT I
210 GOTO 510
220 IF ISO = 15 THEN 230 ELSE 280
230   FOR I = 1 TO 10
240         LET T1(I) = ISO15A(I)
250         LET T2(I) = ISO15B(I)
260   NEXT I
270 GOTO 510
280 IF ISO = 20 THEN 290 ELSE 340
290   FOR I = 1 TO 10
300         LET T1(I) = ISO20A(I)
310         LET T2(I) = ISO20B(I)
320   NEXT I
330 GOTO 510
340 IF ISO = 25 THEN 350 ELSE 400
350   FOR I = 1 TO 10
360         LET T1(I) = ISO25A(I)
370         LET T2(I) = ISO25B(I)
380   NEXT I
390 GOTO 510
400 IF ISO = 30 THEN 410 ELSE 455
410   FOR I = 1 TO 10
420         LET T1(I) = ISO30A(I)
430         LET T2(I) = ISO30B(I)
440   NEXT I
450 GOTO 510
455 IF ISO = 60 THEN 460 ELSE 485
```

```
460   FOR I = 1 TO 10
465         LET T1(I) = ISO60A(I)
470         LET T2(I) = ISO60B(I)
475   NEXT I
480 GOTO 510
485 IF ISO = 90 THEN 490 ELSE 506
490   FOR I = 1 TO 10
495         LET T1(I) = ISO90A(I)
500         LET T2(I) = ISO90B(I)
504   NEXT I
505 GOTO 510
506 PRINT "YOU HAVE CHOSEN A WRONG FIRE EXPOSURE TIME"
507 END

510

REM ************ CALCULATION OF P-DELTA EFFECTS ************
FOR J = 1 TO G
AVDELTA(J) = (DELTA(G + 2 - J) + DELTA(G + 1 - J)) / 2
D(J) = 1000 - 75 + AVDELTA(J)
SWM(J) = D(J) * SW / 1000
NEXT J

REM ************ MAJOR LOOP - DISCRETISATION UP THE WALL ************
520 FOR J = 1 TO G

521 IF J <= K THEN 525 ELSE 522

522   FOR I = 1 TO 10
         LET T1(I) = T2(I)
      NEXT I

525 FOR I = 1 TO 10
526      IF T1(I) < 700 THEN 527 ELSE 529
527      ALPHAC(I) = ((-.00018) + (.000009 * T1(I)) + (2.3E-11 * (T1(I) ^ 3))) / T1(I)
528      GOTO 530
529      ALPHAC(I) = .014 / T1(I)
530 NEXT I

540   FOR I = 1 TO 10
541         IF T1(I) < 750 THEN 542 ELSE 544
542         ALPHAS(I) = ((-.0002416) + (.000012 * T1(I)) + (4E-09 * (T1(I) ^ 2))) / T1(I)
543         GOTO 548
544         IF T1(I) < 860 THEN 545 ELSE 547
545         ALPHAS(I) = .011 / T1(I)
546         GOTO 548
547         ALPHAS(I) = ((-.0062) + (.00002 * T1(I))) / T1(I)
548   NEXT I

FOR I = 1 TO 10
     FC(I) = (-.11122361# + (1.0663556# / (1 + ((T1(I) / 604.16012#) ^ 3.4937958#)))) * 10
     EC(I) = .0025046758# + (.012746344# / (1 + (EXP(-(T1(I) - 448.18143#) / 131.32673#))))
     THSTRAINC(I) = ALPHAC(I) * (T1(I) - 20)
     THSTRAINS(I) = ALPHAS(I) * (T1(I) - 20)
     STMOD(I) = (EXP(9.8979704# + (-.00000011447754# * (T1(I) ^ 2) * (T1(I) ^ (1 / 2))))) * 10
     PRSTRESS(I) = (EXP(3.2218205# + (-.0000047300503# * ((T1(I)) ^ 2)))) * 10
     MSTRESS(I) = (.37559807# + (24.839931# / (1 + (T1(I) / 589.44682#) ^ 7.6729164#))) * 10
     PRSTRAIN(I) = PRSTRESS(I) / STMOD(I)
NEXT I

SWMTOT = SWMTOT + SWM(J)
REM IF J > 1 THEN 1000

REM ************ INITIALISATION FOR NEWTON-RAPHSON SOLVING METHOD ************
FOR AA = .5 TO 1.5 STEP .05
EO1 = THSTRAINC(1) * AA
FOR BB = .5 TO 1.5 STEP .05
COUNTER = 0
EI = THSTRAINC(10) * BB
M1 = (EI - EO1) / 135
```

```
1000 REM ************ MINOR LOOP - TRIAL 1 ************
1005 REM PRINT "EO1 IS"; EO1
1006 REM PRINT "EI IS"; EI
1007 REM PRINT "M1 IS"; M1
1060    FOR I = 1 TO 10
1070        ETOT1(I) = (M1 * (DIST(I) - 7.5)) + EO1
1080        IF I = X THEN 1120 ELSE 1090
1090        STSTRAIN1(I) = THSTRAINC(I) - ETOT1(I)
1100        THSTRAIN(I) = THSTRAINC(I)
1110        GOTO 1140
1120        STSTRAIN1(I) = THSTRAINS(X) - ETOT1(X)
1130        THSTRAIN(I) = THSTRAINS(X)
1140    NEXT I
1150 REM PRINT "BOUNDARY", "TOT STRAIN", "THER STRAIN", "STR STRAIN"
1160    FOR I = 1 TO 10
1170 REM PRINT I, ETOT1(I); ; THSTRAIN(I); ; STSTRAIN1(I)
1180    NEXT I
1190 FOR I = 1 TO 10
1200    IF STSTRAIN1(I) <= 0 THEN 1210 ELSE 1230
1210    IF I = X THEN 1340 ELSE 1600
1230    CONCSTR1(I) = (FC(I) * FCO / 10) * ((STSTRAIN1(I) / EC(I)) * (3 / (2 + ((STSTRAIN1(I) / EC(I)) ^ 3))))
1240    FORCE1(I) = CONCSTR1(I) * AREA / 1000
1250    MOM1(I) = FORCE1(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
1260    GOTO 1600
1340    IF T1(I) < 300 THEN 1350 ELSE 1370
1350    FAT = 1.25 * FAY
1360    GOTO 1410
1370    IF T1(I) >= 300 OR T1(I) < 400 THEN 1380 ELSE 1400
1380    FAT = FAY * (2 - (.0025 * T1(I)))
1390    GOTO 1410
1400    FAT = MSTRESS(I)
1410    IF ABS(STSTRAIN1(I)) < PRSTRAIN(I) THEN 1420 ELSE 1440
1420    FT(I) = -(STMOD(I) * ABS(STSTRAIN1(I)))
1430    GOTO 1580
1440    IF ABS(STSTRAIN1(I)) < .02 THEN 1450 ELSE 1500
1450    C(I) = ((MSTRESS(I) - PRSTRESS(I)) ^ 2) / ((2 * (PRSTRESS(I) - MSTRESS(I))) + (STMOD(I) * (.02 -
PRSTRAIN(I))))
1460    B(I) = ((STMOD(I) * (.02 - PRSTRAIN(I)) * C(I)) + (C(I) ^ 2)) ^ (1 / 2)
1470    A(I) = (((STMOD(I) * ((.02 - PRSTRAIN(I)) ^ 2)) + (C(I) * (.02 - PRSTRAIN(I)))) / STMOD(I)) ^ (1 / 2)
1480    FT(I) = -((B(I) / A(I)) * (((A(I) ^ 2) - ((.02 - (ABS(STSTRAIN1(I)))) ^ 2)) ^ (1 / 2)) + PRSTRESS(I) - C(I))
1490    GOTO 1580
1500    IF ABS(STSTRAIN1(I)) < .04 THEN 1510 ELSE 1530
1510    FT(I) = -((((FAT - MSTRESS(I)) / .02) * STSTRAIN1(I)) - FAT + (2 * MSTRESS(I)))
1520    GOTO 1580
1530    IF ABS(STSTRAIN1(I)) < .15 THEN 1540 ELSE 1560
1540    FT(I) = -FAT
1550    GOTO 1580
1560    IF ABS(STSTRAIN1(I)) < .2 THEN 1570 ELSE 1565
1565    PRINT "YOUR VALUE OF STRESS STRAIN IS NOT LESS THAN 20% - PLEASE CHECK"
1566 GOTO 4445
1570    FT(I) = -(FAT * (1 - ((STSTRAIN1(I) - .15) / .05)))
1580    FORCE1(I) = FT(I) * AST / 1000
1590    MOM1(I) = FORCE1(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
1600 NEXT I
1610 FT1 = FORCE1(1) + FORCE1(2) + FORCE1(3) + FORCE1(4) + FORCE1(5) + FORCE1(6) + FORCE1(7) +
FORCE1(8) + FORCE1(9) + FORCE1(10)
1620 MT1 = MOM1(1) + MOM1(2) + MOM1(3) + MOM1(4) + MOM1(5) + MOM1(6) + MOM1(7) + MOM1(8) + MOM1(9) +
MOM1(10)
1630 REM PRINT
1640 REM PRINT
1650 REM PRINT "BOUNDARY", "FORCE (kN)", "MOMENT (kN.m)"
1660 FOR I = 1 TO 10
1670 REM PRINT I, FORCE1(I), MOM1(I)
1680 NEXT I
1690 REM PRINT
1700 REM PRINT
1730 FR = N + (SW * J)
1735 MR = (N * ((DELTA(G + 1)) + 1000 - 75) / 1000) + SWMTOT - LATMOM(J)
1736 REM PRINT "FR IS"; FR
1737 REM PRINT "MR IS"; MR
1738 REM PRINT
1740 TOTFOR1 = FT1 - FR
```

```
1745 TOTMOM1 = MT1 - MR
1746 REM PRINT "TOTAL FORCE ASSOCIATED WITH TRIAL 1 IS"; TOTFOR1
1747 REM PRINT "TOTAL MOMENT ASSOCIATED WITH TRIAL 1 IS"; TOTMOM1
1748 REM PRINT
1749 REM PRINT "SWMTOT IS"; SWMTOT
1750 REM PRINT
1751 IF ABS(TOTFOR1) > 1 THEN 2000 ELSE 1755
1755 IF ABS(TOTMOM1) > 1 THEN 2000 ELSE 4544


2000 REM ************ MINOR LOOP - TRIAL 2 ************
2010 EO2 = EO1
2020 M2 = M1 + (M1 / 1000000)
2030 REM PRINT "EO2 IS"; EO2
2040 REM PRINT "EI IS"; EI
2050 REM PRINT "M2 IS"; M2
2060 FOR I = 1 TO 10
2070         ETOT2(I) = (M2 * (DIST(I) - 7.5)) + EO2
2080         IF I = X THEN 2120 ELSE 2090
2090         STSTRAIN2(I) = THSTRAINC(I) - ETOT2(I)
2100         THSTRAIN(I) = THSTRAINC(I)
2110         GOTO 2140
2120         STSTRAIN2(I) = THSTRAINS(X) - ETOT2(X)
2130         THSTRAIN(I) = THSTRAINS(X)
2140    NEXT I
2150 REM PRINT "BOUNDARY", "TOT STRAIN", "THERM STRAIN", "STR STRAIN"
2160    FOR I = 1 TO 10
2170 REM PRINT I, ETOT2(I); ; THSTRAIN(I); ; STSTRAIN2(I)
2180    NEXT I
2190 FOR I = 1 TO 10
2200    IF STSTRAIN2(I) <= 0 THEN 2210 ELSE 2230
2210    IF I = X THEN 2340 ELSE 2600
2230    CONCSTR2(I) = (FC(I) * FCO / 10) * ((STSTRAIN2(I) / EC(I)) * (3 / (2 + ((STSTRAIN2(I) / EC(I)) ^ 3))))
2240    FORCE2(I) = CONCSTR2(I) * AREA / 1000
2250    MOM2(I) = FORCE2(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
2260    GOTO 2600
2340    IF T1(I) < 300 THEN 2350 ELSE 2370
2350    FAT = 1.25 * FAY
2360    GOTO 2410
2370    IF T1(I) >= 300 OR T1(I) < 400 THEN 2380 ELSE 2400
2380    FAT = FAY * (2 - (.0025 * T1(I)))
2390    GOTO 2410
2400    FAT = MSTRESS(I)
2410    IF ABS(STSTRAIN2(I)) < PRSTRAIN(I) THEN 2420 ELSE 2440
2420    FT(I) = -(STMOD(I) * ABS(STSTRAIN2(I)))
2430    GOTO 2580
2440    IF ABS(STSTRAIN2(I)) < .02 THEN 2450 ELSE 2500
2450    C(I) = ((MSTRESS(I) - PRSTRESS(I)) ^ 2) / ((2 * (PRSTRESS(I) - MSTRESS(I))) + (STMOD(I) * (.02 -
PRSTRAIN(I))))
2460    B(I) = ((STMOD(I) * (.02 - PRSTRAIN(I)) * C(I)) + (C(I) ^ 2)) ^ (1 / 2)
2470    A(I) = (((STMOD(I) * ((.02 - PRSTRAIN(I)) ^ 2)) + (C(I) * (.02 - PRSTRAIN(I)))) / STMOD(I)) ^ (1 / 2)
2480    FT(I) = -((B(I) / A(I)) * (((A(I) ^ 2) - ((.02 - (ABS(STSTRAIN2(I)))) ^ 2)) ^ (1 / 2)) + PRSTRESS(I) - C(I))
2490    GOTO 2580
2500    IF ABS(STSTRAIN2(I)) < .04 THEN 2510 ELSE 2530
2510    FT(I) = -((((FAT - MSTRESS(I)) / .02) * STSTRAIN2(I)) - FAT + (2 * MSTRESS(I)))
2520    GOTO 2580
2530    IF ABS(STSTRAIN2(I)) < .15 THEN 2540 ELSE 2560
2540    FT(I) = -FAT
2550    GOTO 2580
2560    IF ABS(STSTRAIN2(I)) < .2 THEN 2570 ELSE 2565
2565    PRINT "YOUR VALUE OF STRESS STRAIN IS NOT LESS THAN 20% - PLEASE CHECK"
2566    GOTO 4445
2570    FT(I) = -(FAT * (1 - ((STSTRAIN2(I) - .15) / .05)))
2580    FORCE2(I) = FT(I) * AST / 1000
2590    MOM2(I) = FORCE2(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
2600 NEXT I
2610 FT2 = FORCE2(1) + FORCE2(2) + FORCE2(3) + FORCE2(4) + FORCE2(5) + FORCE2(6) + FORCE2(7) +
FORCE2(8) + FORCE2(9) + FORCE2(10)
2620 MT2 = MOM2(1) + MOM2(2) + MOM2(3) + MOM2(4) + MOM2(5) + MOM2(6) + MOM2(7) + MOM2(8) + MOM2(9) +
MOM2(10)
2630 REM PRINT
2640 REM PRINT
```

```
2650 REM PRINT "BOUNDARY", "FORCE (kN)", "MOMENT (kN.m)"
2660 FOR I = 1 TO 10
2670 REM PRINT I, FORCE2(I), MOM2(I)
2680 NEXT I
2690 REM PRINT
2700 REM PRINT
2725 TOTFOR2 = FT2 - FR
2730 TOTMOM2 = MT2 - MR
2735 REM PRINT "TOTAL FORCE ASSOCIATED WITH TRIAL 2 IS"; TOTFOR2
2740 REM PRINT "TOTAL MOMENT ASSOCIATED WITH TRIAL 2 IS"; TOTMOM2


3000 REM ************ MINOR LOOP - TRIAL 3 ************
3010 EO3 = EO1 + (EO1 / 1000000)
3020 M3 = M1
3030 REM PRINT "EO3 IS"; EO3
3040 REM PRINT "EI IS"; EI
3050 REM PRINT "M3 IS IS"; M3
3060   FOR I = 1 TO 10
3070          ETOT3(I) = (M3 * (DIST(I) - 7.5)) + EO3
3080          IF I = X THEN 3120 ELSE 3090
3090          STSTRAIN3(I) = THSTRAINC(I) - ETOT3(I)
3100          THSTRAIN(I) = THSTRAINC(I)
3110          GOTO 3140
3120          STSTRAIN3(I) = THSTRAINS(X) - ETOT3(X)
3130          THSTRAIN(I) = THSTRAINS(X)
3140   NEXT I
3150 REM PRINT "BOUNDARY", "TOT STRAIN", "THER STRAIN", " STR STRAIN "
3160    FOR I = 1 TO 10
3170 REM PRINT I, ETOT3(I); ; THSTRAIN(I); ; STSTRAIN3(I)
3180    NEXT I
3190 FOR I = 1 TO 10
3200    IF STSTRAIN3(I) <= 0 THEN 3210 ELSE 3230
3210    IF I = X THEN 3340 ELSE 3600
3230    CONCSTR3(I) = (FC(I) * FCO / 10) * ((STSTRAIN3(I) / EC(I)) * (3 / (2 + ((STSTRAIN3(I) / EC(I)) ^ 3))))
3240    FORCE3(I) = CONCSTR3(I) * AREA / 1000
3250    MOM3(I) = FORCE3(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
3260    GOTO 3600
3340    IF T1(I) < 300 THEN 3350 ELSE 3370
3350    FAT = 1.25 * FAY
3360    GOTO 3410
3370    IF T1(I) >= 300 OR T1(I) < 400 THEN 3380 ELSE 3400
3380    FAT = FAY * (2 - (.0025 * T1(I)))
3390    GOTO 3410
3400    FAT = MSTRESS(I)
3410    IF ABS(STSTRAIN3(I)) < PRSTRAIN(I) THEN 3420 ELSE 3440
3420    FT(I) = -(STMOD(I) * ABS(STSTRAIN3(I)))
3430    GOTO 3580
3440    IF ABS(STSTRAIN3(I)) < .02 THEN 3450 ELSE 3500
3450    C(I) = ((MSTRESS(I) - PRSTRESS(I)) ^ 2) / ((2 * (PRSTRESS(I) - MSTRESS(I))) + (STMOD(I) * (.02 -
PRSTRAIN(I))))
3460    B(I) = ((STMOD(I) * (.02 - PRSTRAIN(I)) * C(I)) + (C(I) ^ 2)) ^ (1 / 2)
3470    A(I) = (((STMOD(I) * ((.02 - PRSTRAIN(I)) ^ 2)) + (C(I) * (.02 - PRSTRAIN(I)))) / STMOD(I)) ^ (1 / 2)
3480    FT(I) = -((B(I) / A(I)) * (((A(I) ^ 2) - ((.02 - (ABS(STSTRAIN3(I)))) ^ 2)) ^ (1 / 2)) + PRSTRESS(I) - C(I))
3490    GOTO 3580
3500    IF ABS(STSTRAIN3(I)) < .04 THEN 3510 ELSE 3530
3510    FT(I) = -((((FAT - MSTRESS(I)) / .02) * STSTRAIN3(I)) - FAT + (2 * MSTRESS(I)))
3520    GOTO 3580
3530    IF ABS(STSTRAIN3(I)) < .15 THEN 3540 ELSE 3560
3540    FT(I) = -FAT
3550    GOTO 3580
3560    IF ABS(STSTRAIN3(I)) < .2 THEN 3570 ELSE 3565
3565    PRINT "YOUR VALUE OF STRESS STRAIN IS NOT LESS THAN 20% - PLEASE CHECK"
3566    GOTO 4445
3570    FT(I) = -(FAT * (1 - ((STSTRAIN3(I) - .15) / .05)))
3580    FORCE3(I) = FT(I) * AST / 1000
3590    MOM3(I) = FORCE3(I) * (1000 + DELTA(G + 2 - J) - DIST(I)) / 1000
3600 NEXT I
3610 FT3 = FORCE3(1) + FORCE3(2) + FORCE3(3) + FORCE3(4) + FORCE3(5) + FORCE3(6) + FORCE3(7) +
FORCE3(8) + FORCE3(9) + FORCE3(10)
3620 MT3 = MOM3(1) + MOM3(2) + MOM3(3) + MOM3(4) + MOM3(5) + MOM3(6) + MOM3(7) + MOM3(8) + MOM3(9) +
MOM3(10)
```

```
3630 REM PRINT
3640 REM PRINT
3650 REM PRINT "BOUNDARY", "FORCE (kN)", "MOMENT (kN.m)"
3660 FOR I = 1 TO 10
3670 REM PRINT I, FORCE3(I), MOM3(I)
3680 NEXT I
3690 REM PRINT
3700 REM PRINT
3725 TOTFOR3 = FT3 - FR
3730 TOTMOM3 = MT3 - MR
3735 REM PRINT "TOTAL FORCE ASSOCIATED WITH TRIAL 3 IS"; TOTFOR3
3740 REM PRINT "TOTAL MOMENT ASSOCIATED WITH TRIAL 3 IS"; TOTMOM3


4000 REM ************ NEWTON-RAPHSON METHOD FOR SOLVING A TWO DIMENSIONAL ARRAY ************
4011 DFDE = (FT3 - FT1) / (EO1 / 1000000)
4020 DFDC = (FT2 - FT1) / (M1 / 1000000)
4030 DMDE = (MT3 - MT1) / (EO1 / 1000000)
4040 DMDC = (MT2 - MT1) / (M1 / 1000000)
4050 DET = (DFDE * DMDC) - (DFDC * DMDE)
4060 REM PRINT "THE VALUE OF THE DETERMINANT IS"; DET
4070 REM PRINT
4080 REM PRINT "FT1 IS"; FT1
4090 REM PRINT "FT2 IS"; FT2
4100 REM PRINT "FT3 IS"; FT3
4110 REM PRINT "MT1 IS"; MT1
4120 REM PRINT "MT2 IS"; MT2
4130 REM PRINT "MT3 IS"; MT3
4140 REM PRINT
4150 REM PRINT "DFDE IS"; DFDE
4160 REM PRINT "DFDC IS"; DFDC
4170 REM PRINT "DMDE IS"; DMDE
4180 REM PRINT "DMDC IS"; DMDC
4190 REM PRINT
4200 REM PRINT "EXT. STRAIN", "CURVATURE", "FORCE", "MOMENT"
4210 REM PRINT

4221 IF M1 < 0 THEN GOTO 4222 ELSE 4250
4222 PRINT "M1 IS LESS THAN ZERO"
4223 GOTO 4445
4250 EO1NEW = EO1 + (((-DMDC * TOTFOR1) + (DFDC * TOTMOM1)) / DET)
4260 PHINEW = M1 + (((DMDE * TOTFOR1) - (DFDE * TOTMOM1)) / DET)

4360 EO1 = EO1NEW
4370 M1 = PHINEW
4380 FT1 = 0
4381 FT2 = 0
4382 FT3 = 0
4383 MT1 = 0
4384 MT2 = 0
4385 MT3 = 0
4400 FOR I = 1 TO 10
4401   FORCE1(I) = 0
4402   FORCE2(I) = 0
4403   FORCE3(I) = 0
4404   MOM1(I) = 0
4405   MOM2(I) = 0
4406   MOM3(I) = 0
4425 NEXT I

4430 COUNTER = COUNTER + 1
REM PRINT "COUNTER IS"; COUNTER
4440 IF COUNTER > 50 THEN 4450 ELSE 4490
4445 EO1 = THSTRAINC(1) * AA
4450 NEXT BB
4460 NEXT AA
4480 PRINT "THERE IS NO SOLUTION FOR THIS RANGE"
4485 END

4490 GOTO 1005

4544 CURV(J) = M1
```

```
4545 PPSI(J) = CURV(J) * L

PRINT "FOUND SOLUTION FOR BOUNDARY"; J

4548 NEXT J


REM ************ CALCULATION OF DEFORMED SHAPE **********
CLS
4554 FOR J = 1 TO G + 1
4556 PPSI(1) = 0
4557 THETA(J) = THETA(J - 1) - PPSI(J)
4558 DISP(J) = INT(DISP(J - 1) + (L * THETA(J - 1)) - (PPSI(J) * L / 2))
4559 PRINT "DISPLACEMENT AT BOUNDARY"; J; "IS"; " "; , -DISP(J)
4560 LET DELTA(J) = -DISP(J)
4570 NEXT J
4575 PRINT

OPEN "B:\DELTA.DAT" FOR OUTPUT AS #3
FOR J = 1 TO G + 1
WRITE #3, DELTA(J)
NEXT J
CLOSE #3


REM ****** DATA LINES CONTAINING ISO FIRE EXPOSURES, CENTROID DISTANCES AND DEFORMED SHAPE ****
REM ****** 2.5 MIN ISO FIRE ******
DATA 25,25,25,25,25,25,25,25,26,90
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 5 MIN ISO FIRE ******
DATA 25,25,25,25,25,25,25,27,57,178
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 7.5 MIN ISO FIRE ******
DATA 25,25,25,25,25,25,26,42,82,249
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 10 MIN ISO FIRE ******
DATA 25,25,25,25,25,26,35,54,125,324
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 15 MIN ISO FIRE ******
DATA 25,25,25,25,28,33,52,88,196,441
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 20 MIN ISO FIRE ******
DATA 25,25,26,28,34,45,71,138,267,506
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 25 MIN ISO FIRE ******
DATA 26,27,28,32,42,59,90,169,320,566
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 30 MIN ISO FIRE ******
DATA 27,28,31,38,50,72,120,209,361,610
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 60 MIN ISO FIRE ******
DATA 47,53,64,82,116,172,254,372,535,766
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** 90 MIN ISO FIRE ******
DATA 85,91,104,137,191,259,351,472,633,850
DATA 25,25,25,25,25,25,25,25,25,25

REM ****** CENTROID DISTANCES ******
DATA 7.5, 22.5, 37.5, 52.5, 67.5, 82.5, 97.5, 112.5, 127.5, 142.5

9999 END
```

APPENDIX D EFFECTS OF INCREASED WALL DISCRETISATION

D1.0 Introduction

As mentioned in Chapter 3, the wall model may be discretised into any number of elements across the wall thickness and up the wall height. The model was able to directly cater for varying levels of discretisation through user input; ie. the wall model program is fully flexible and sensitive to user input.

This appendix looks at investigating the effect that increased wall discretisation has on the overall accuracy of results. Only the effect of increased discretisation up the height of the wall is considered, previous analyses having established the suitability of the degree of discretisation across the wall thickness.

D2.0 Case Considered

The case considered to demonstrate the above effects is described as follows:

Wall Height:	6 m	
Effective Wall Width:	1 m	
Wall Thickness:	150 mm	
Fire Exposure:	ISO 384 - over entire height of wall	
Fire Exposure Time:	10 minutes	
Wall Reinforcement:	F82 Mesh centrally located in wall	
Reinforcement Ratio:	(i)	0.0015; minimum reinforcement
	(ii)	0.0030; double reinforcement
Concrete Compressive Strength:	32 MPa	
Reinforcing Steel Yield Strength:	400 MPa	

Two reinforcement situations are considered - the minimum reinforcement in a single layer, and twice the minimum arranged in two layers. The reinforcement ratio of 0.0015 translates to approximately 225 mm^2/m of F82 mesh whilst the ratio of 0.0030 is approximately 450 mm^2/m.

The above estimate of minimum reinforcement is consistent with that of current designs and complies with AS 3600 (1988) - Section 11.

D3.0 Method of Analysis

The above wall examples were arbitrarily divided into 2, 5, 15, 20 and 25 elements. This spread of elements was seen to give a reasonable indication of the effects that increased discretisation has on the magnitude of displaced shape, and therefore of the calculated fire-resistance.

For the purpose of this study the fire exposure was assumed to act over the entire wall height thus subjecting each of the elements up the wall height to identical thermal gradients.

This study considered the effects of fire only, with no lateral or axial loads being included, with the exception of self weight.

D4.0 Results of Analysis

The results of the above analysis are summarised in Table D.1 below.

Table D.1 **Maximum Deflection of 6 m Wall with Varying Levels of Discretisation**

MAXIMUM DEFLECTION AT TOP OF 6 m WALL, (mm)					
Reinforcement Ratio, p_w	Level of Discretisation				
	Number of Elements Up Length of Wall				
	2	*5*	*15*	*20*	*25*
0.0015	326	420	439	438	439
0.0030	323	409	406	405	399

It can be seen from the above results that the number of elements chosen will effect the accuracy of the results. However, even five elements gives reasonable accuracy and beyond fifteen there is no gain in accuracy. The conclusion is true for both reinforcement cases.

This conclusion is true for both reinforcement cases.

APPENDIX E CONNECTION MATERIAL PROPERTY TESTS

E1.0 Introduction

As previously mentioned, the steel components and concrete associated with the tested connections were individually tested and recorded. This Appendix presents the results of these tests.

E2.0 Concrete Component Tests

At the time the slabs were poured, five cylinders were cast to determine the compressive strength and stress-strain relationships for the concrete.

Below is a summary of the compressive strength tests.

Table E.1 Concrete Compressive Test Results - Pretest Data

Concrete Property	Age at Time of Test		
	21 Days	28 Days	35 Days
Expected Concrete Strength; (MPa)	-	32	-
Actual Concrete Strength; (MPa)	48.5	45.8	49.4
Ultimate Concrete Load; (kN)	855	805	872
Density; (kg/m^3)	2426	2447	2415
Cylinder Height; (mm)	299	299	299
Cylinder Diameter 1; (mm)	150.4	150.2	150.1
Cylinder Diameter 2; (mm)	149.2	150.0	149.7
Cylinder Weight; (kg)	12.783	12.947	12.745

Two tests were conducted on the concrete cylinders in accordance with AS1012, Part 17 - 1976 to obtain 'typical' stress-strain relationships. The setup is shown below in Figure E.1.

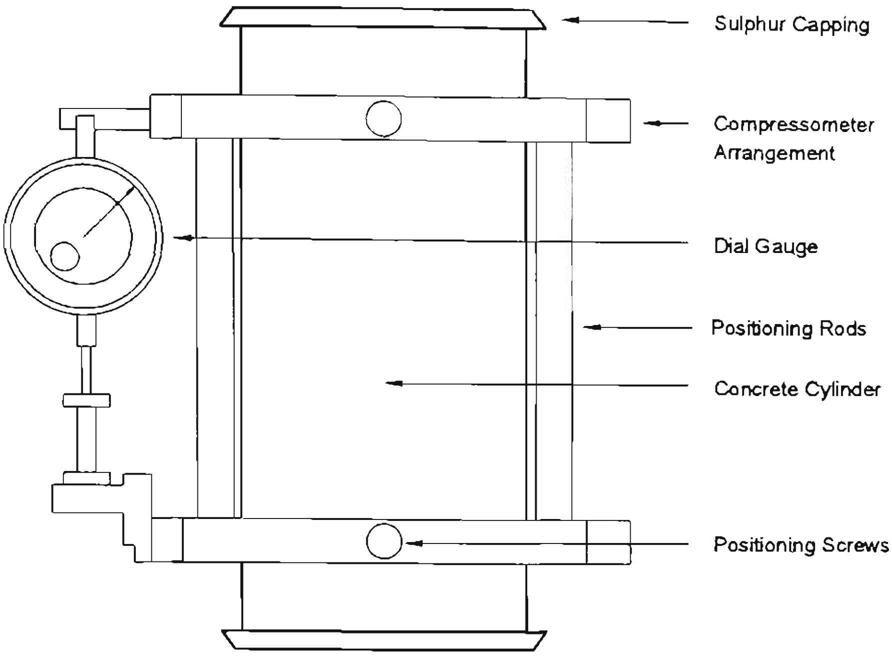


Figure E.1 Typical Compressometer Arrangement for Measurement of Longitudinal Strain

The stress-strain relationships are given below.

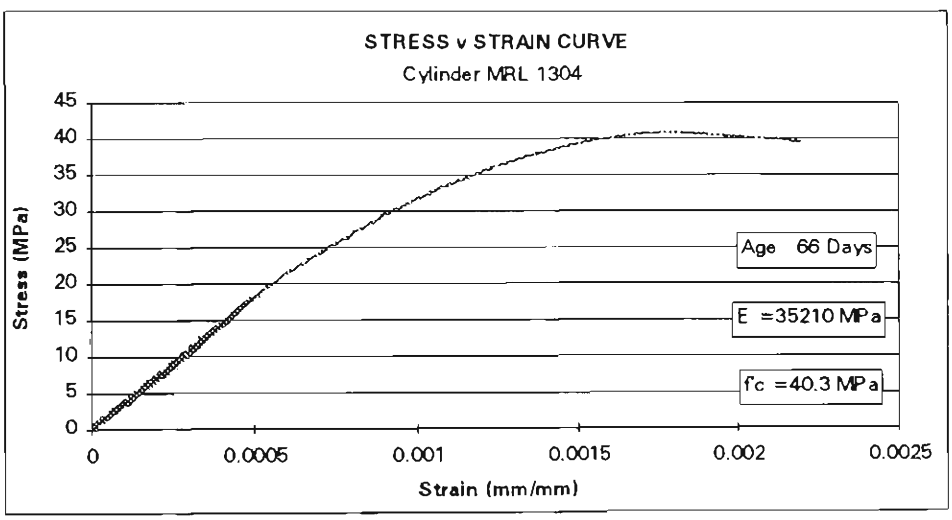


Figure E.2 Stress-Strain Relationship for Cylinder MRL 1304

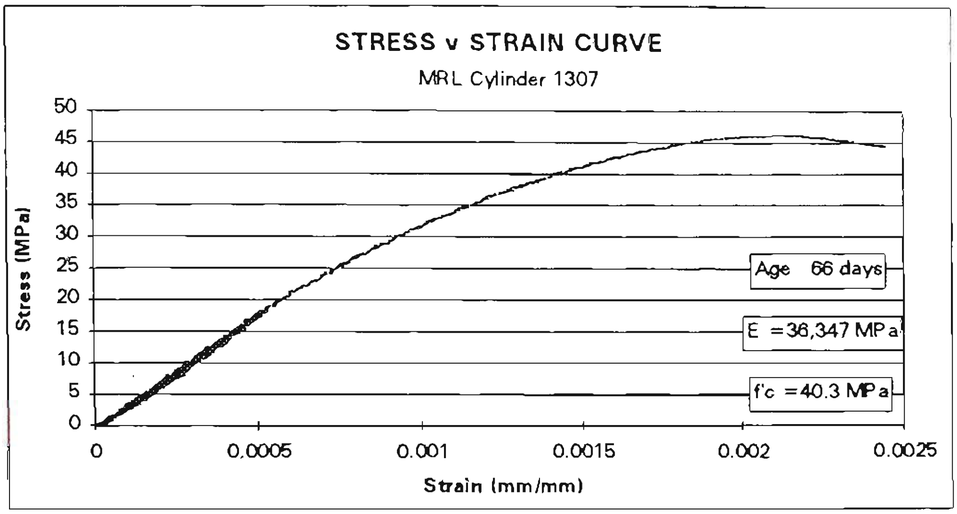


Figure E.3 Stress-Strain Relationship for Cylinder MRL 1307

E3.0 Steel Component Tests

To determine the Modulus of Elasticity of the steel plate used in the connection tests, a number of room temperature tensile tests had to be undertaken. In all, three coupons were cut from the same specimen of steel used for the connection tests. The dimensions of the steel coupon are given in Figure E4 below.

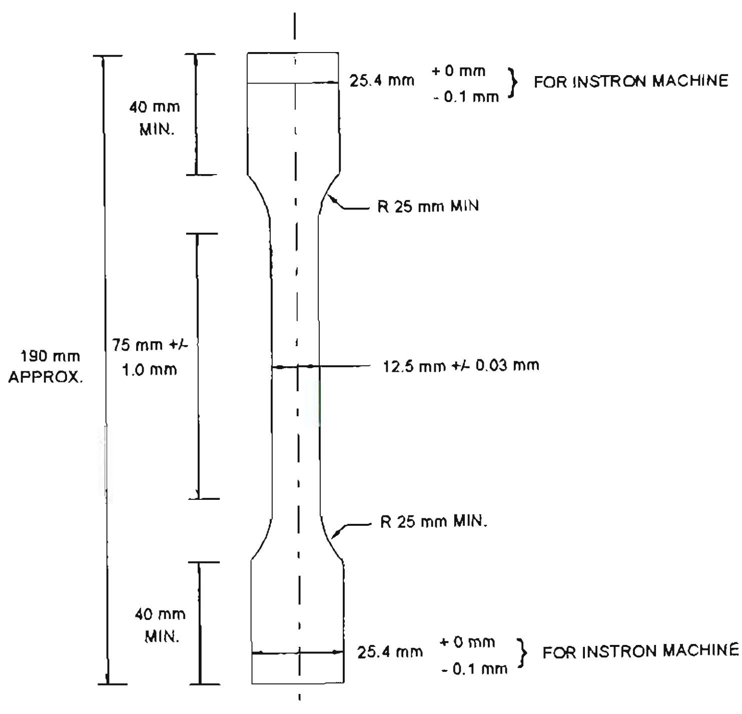


Figure E.4 Standard B-34 Tensile Coupon Test Specimen

The stress-strain relationships obtained from the steel coupon tests are given below.

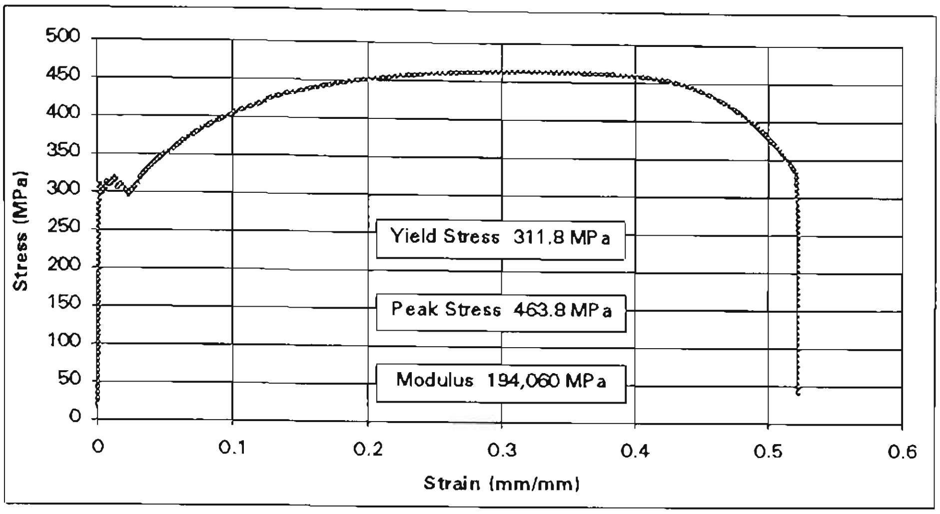


Figure E.5 Stress-Strain Relationship for Coupon Test 1

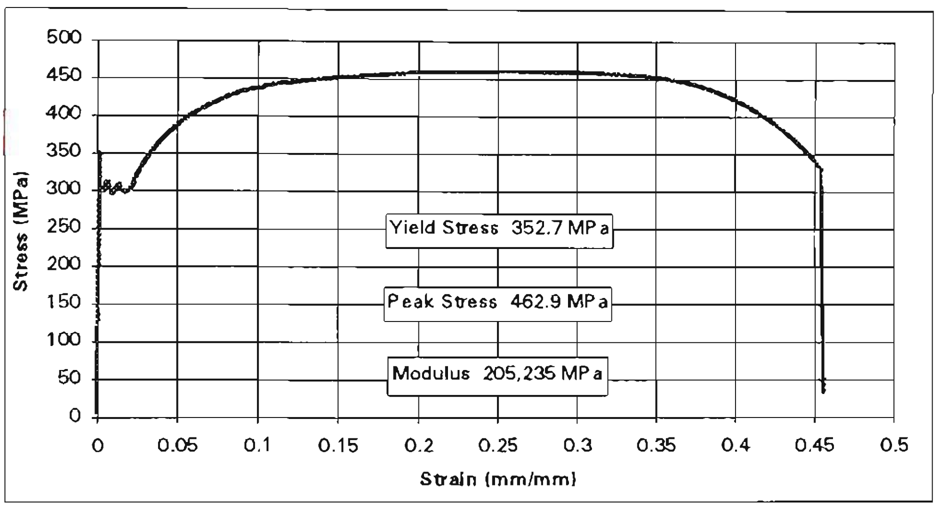


Figure E.6 Stress-Strain Relationship for Coupon Test 2

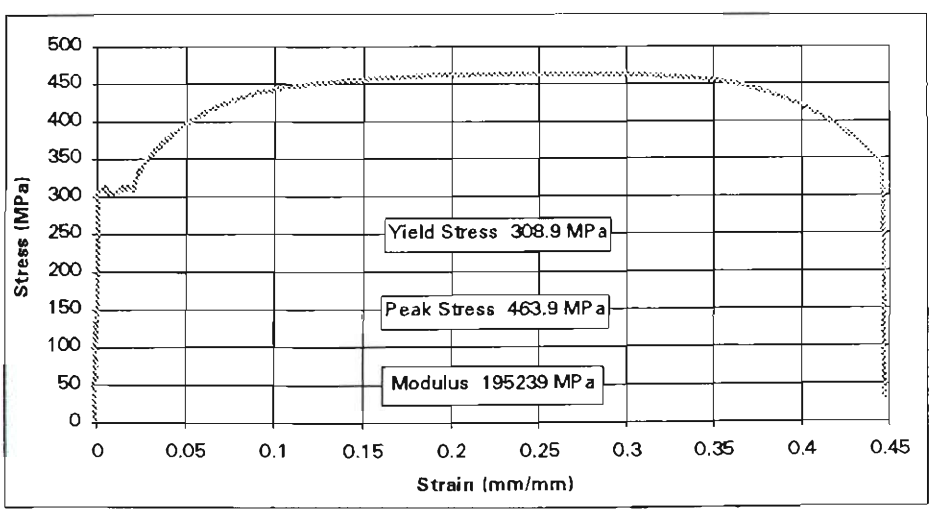


Figure E.7 Stress-Strain Relationship for Coupon Test 3

APPENDIX F DETERMINATION OF APPROXIMATE CLIP PLATE CAPACITIES

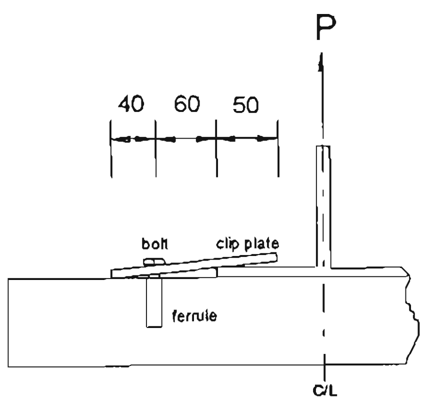
F1.0 Introduction

As previously mentioned, determination of the clip plate capacities is important for comparison with the results obtained from the experimental tests as reported in Section 5.6.

The strength of two types of clip plate connection, as shown in Figure 5.3, are determined in this Appendix.

F2.0 Calculation of Clip Plate Strength

F2.1 Non-Welded Clip Plate



UN-WELDED CLIP CONNECTION

Figure F.1 Un-Welded Clip Plate Connection

(A) Clip Plate Capacity

Equating the external forces with the internal forces experienced by the connection the following is obtained:

Internal Moment: $$M_i = \frac{bd^2}{4} \cdot f_y \qquad \text{Eq. F1}$$

External Moment: $$M_e = \frac{P}{2} \cdot l_a \qquad \text{Eq. F2}$$

where:

M_i = internal moment

M_e = external moment

b = clip plate width

d = clip plate thickness

f_y = steel yield strength

P = ultimate applied connection load

l_a = lever arm between bolt and flange edge

Equating the above equations and solving for P:

$$P = \frac{bd^2 \cdot f_y}{2l_a} \qquad \text{Eq. F3}$$

$$P = \frac{100 \times 10^2 \cdot 280}{2 \times 60}$$

$$P = 23.3 \text{ kN}$$

The total load able to be carried by each connection is 23.3 kN

(B) Bolt Capacity

The design bolt capacity can be determined from the AISC - Design Capacity Tables for Structural Steel, (1st Edition - 1991). From Table 7.1.3 (b) of the Design Capacity Tables;

Design Capacity for M16 High Strength bolt in Axial Tension is 104 kN

Clearly for the clip plate to act as a cantilever in bending, some prying effects must be present (Figure F.2).

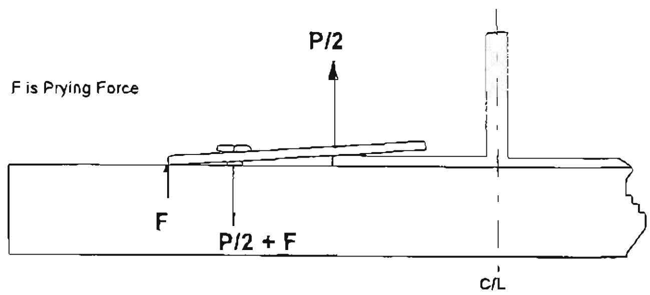


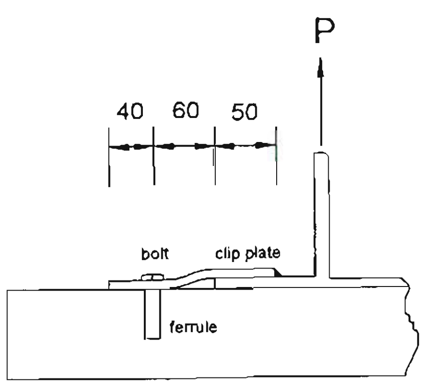
Figure F.2 Development of Prying Force

The force in the bolt is larger than P / 2 and therefore the total capacity of the connection with respect to bolt capacity is less than the 104 kN.

(C) Ferrule Capacity

From the manufacturer's design tables, Ramset indicate that the ultimate tensile load that the M16 ferrules can resist is approximately 54 kN.

F2.2 Welded Clip Plate



ELEVATION

WELDED CLIP CONNECTION

Figure F.3 Welded Clip Plate Connection

(A) Clip Plate Capacity

Equating the external forces with the internal forces experienced by the connection we obtain the following:

Internal Moment: $$2M_i = 2 \cdot \frac{bd^2}{4} \cdot f_y \qquad \text{Eq. F4}$$

External Moment: $$M_e = \frac{P}{2} \cdot l_a \qquad \text{Eq. F5}$$

where:

M_i = internal moment	f_y = steel yield strength
M_e = external moment	P = ultimate applied connection load
b = clip plate width	l_a = lever arm between bolt and flange edge
d = clip plate thickness	

Equating the above equations and solving for P we get:

$$P = \frac{bd^2 \cdot f_y}{l_a} \qquad \text{Eq. F6}$$

$$P = \frac{100 \times 10^2 \cdot 280}{60}$$

$$P = 46.7 \text{ kN}$$

Therefore each connection can withstand a force, with respect to plate bending of 46.7 kN.

(B) Bolt Capacity

See discussion (B) in F2.1

(C) Ferrule Capacity

See discussion (C) in F2.1

(D) Weld Capacity

From the AISC - Design Capacity Tables for Structural Steel, (1st Edition), the design weld capacity can be determined. From Table 7.2.4-1 (b) of the Design Capacity Tables;

The weld was designed to resist the expected failure load of the ferrule; ie. the weakest link in the connection was considered to be the ferrule.

For a 6 mm, structural purpose (SP) weld and using a E48XX/W50X electrode then the required weld was approximately equal to 2 lengths of 53 mm; ie. 2/53 mm.