Malcolm J Davis

From: Ian Bennetts [ian.bennetts@noel-arnold.com.au]
Sent: Monday, 15 March 2010 10:05 AM
To: Malcolm Davis
Subject: Proposed Modification for Spec C1.11

Noel Arnold & Associates
Risk Management Consultants

Dear Malcolm

Thanks for sending through the package of material. I have read the additional clause which has been proposed to be added to Specification C1.11. As I understand it, you are proposing to add an additional part to Clause 3 of Specification C1.11 to require that "where clips are used to provide lateral support to a panel, these are designed so they cannot rotate and therefore become detached from the supported panel, either prior to or during a fire".

I would suggest that the above wording (or something like it) is used rather than that proposed in the current "Proposal for Change" document and that it is added as subclause (g) of clause 3 of Specification C1.11.

I consider that the proposed addition is fully justified from a safety perspective since there are instances where clips are no longer welded and where rotation occurs even prior to a fire event. The proposed addition does not require welding but still permits this or any other method of achieving the desired outcome – including the one proposed by your company.

Kind Regards

Ian Bennetts
Principal Risk Consultant
Noel Arnold & Associates
Phone: (03) 9890 8811 Fax: (03) 9890 8911
Direct: (03) 9896 8709
Mobile: 0400 062 899
Email: ian.bennetts@noel-arnold.com.au
www.noel-arnold.com.au

Melbourne Office	Sydney Office	Canberra Office	Brisbane Office
Level 3 / 818 Whitehorse Rd	Level 2 / 11 Khartoum Road	Unit 21 / Level 3 Belconnen	Level 1 / 2 Mayneview St
Box Hill VIC 3128	North Ryde NSW 2113	Churches Centre, Benjamin Way	Milton QLD 4066
Tel: 03 9890 8811	Tel: 02 9889 1800	Belconnen ACT 2617	Tel: 07 3514 9222
Fax: 03 9890 8911	Fax: 02 9889 1811	Tel: 02 9889 1800	Fax: 07 3514 9220
		Fax: 02 9889 1811	

Ian Bennetts *is a Professorial Fellow at Victoria University of Technology's Centre for Environmental Safety and Risk Engineering (CESARE) where he heads a research team which undertakes research for OneSteel, BHP and other industrial partners. He is also involved with teaching post-graduate courses in fire engineering at the University. His research interests include the behaviour of building structures in fire, fire-safety engineering and risk management. He has published more than 100 papers, reports and book contributions on many aspects of fire, construction and risk engineering; and has been a fire-engineering consultant on major projects such as the Melbourne Casino, Federation Square, Brisbane and Adelaide International Airport Terminals and the Brisbane Cricket Ground.*

IAN BENNETTS

Engineer with experience in the behaviour of steel, composite and concrete structures under ambient and fire conditions. He also has experience in risk assessment and in all aspects of fire-safety engineering. He contributes to the postgraduate diploma course at Victoria University and has close links with the university.

Prior to joining Noel Arnold & Associates, Ian was a Professorial Fellow at the Centre for Environmental Safety and Risk Engineering at Victoria University, Melbourne, where he was involved in the teaching of fire-engineering subjects for the postgraduate certificate and diploma courses as well as supervising and undertaking research in a number of areas – the behaviour of structures in fire, modelling of fires, reliability of sprinkler systems, fire safety of shopping centres, effectiveness of stair pressurisation systems – as well as setting up a test rig for the testing of fire fighting foams at VU's Fiskville test facility.

Before joining Victoria University in 1999, Ian spent 19 years at BHP Melbourne Research Laboratories reaching the position of Senior Research Associate and becoming responsible for structural engineering and fire research at these laboratories. He was also a NATA signatory for heat and temperature measurement and mechanical testing. During that time Ian contributed to Fire Code Reform Centre Projects, leading Project 6 'Fire Safety of Shopping Centres' and being involved in aspects of Project 2 (materials) and Project 3 (fire resistance). He was also involved in research and testing relating to the fire safety of carparks, office buildings and elements providing support to other elements. The outcomes from this research and testing resulted in significant changes to deemed-tosatisfy provisions of the Building Code of Australia.

He has published more than 120 research papers and reports and has made contributions to numerous books including Risk Analysis in Building Fire Safety Engineering by Hasofer, Beck and Bennetts and published by Elsevier in November 2006.

Dr. Ian Bennetts Joins NAA Fire Engineering Team

The NAA fire-engineering team has a valued and experienced new member.

Over the past six years, Dr Ian Bennetts has made an important contribution to the Centre for Environmental Safety and Risk Engineering (CESARE) at Victoria University, which was the first Australian tertiary institution to offer formal qualifications in fire engineering.

Ian's career spans 20 years in structural and fire engineering at BHP. He has worked in design, consulting and research and has published more than 120 professional papers and reports. He has developed and published building fire-engineering guides for fire safety at shopping centres and other sites. Ian regularly provides peer review for construction firms and expert opinion in legal matters.

With Ian's appointment, Noel Arnold & Associate's team of fire-risk engineers will continue the relationship with Victoria University, providing lecturing and input into its courses. Access to the university's testing laboratories will also benefit NAA's clients.

Technical Note Issue no.1 March 1995
BHP Structural Steel Development Group

Single Storey Steel-framed Buildings

Support of External Walls in Fire

Authors: Dr Ian Bennetts and Dr Tony O'Meagher

1. Introduction

The Building Code of Australia (BCA) [1] states 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 external walls that could collapse as complete panels (eg. tilt-up and precast concrete), they must be designed so that in the event of a fire the likelihood of outward collapse of the panels is minimised.

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

Clause C1.11 has been included in the BCA as the result of situations where fires have occurred in single storey warehouse buildings and external wall panels have become detached during the fire and fallen outwards. These failures have been largely due to the poor design and detailing of the connections between the supporting structure and the panels. The outwards collapse of wall panels is considered to be undesirable as it may lead to damage of adjacent property, or to persons stationed external to the building during the fire, such as fire fighters.

The purpose of this technical note is to provide design details which satisfy the requirements of Clause C1.11 for buildings incorporating steel portal frames or steel beams and columns, as illustrated in Figure 1. The engineering basis for the proposed details is also described. The details presented in this technical note are not the only details which will satisfy the Clause C1.11 but represent several practical solutions.

As will be noted from Figure 1, buildings may be clad with horizontal or vertical wall panels.

For the purpose of this publication, *horizontal panels* are defined as those which resist lateral wind loads predominantly through bending action between the supporting columns, whilst *vertical panels* are those which resist lateral wind loads mainly through bending action over the height of the storey. As shown in Figure 1 concrete wall panels may be large singular panels, interlocking panels, or separate individual panels.

In the case of the *side walls* of a building, vertical panels are generally supported laterally at the top by an eaves tie member (see Figure 2), whilst in the case of the end *walls*, vertical panels are most likely connected to a raker member which is also connected to the purlins (Figure 3(a)). If there are substantial openings, or horizontal panels, the end walls will probably incorporate a lightweight steel frame of mullions and rakers (Figure3(b)).

2. Support of Vertical Panels - Side Walls

2.1 Background

The behaviour in fire of a steel-framed building with vertical side wall panels is now considered in relation to the stages of fire development and growth.

2.1.1 Early Heating

The development of a fire within a building will result in heating of the inside faces of the wall panels and the development of significant thermal gradients across the thickness of the wall panels. This thermal gradient will give rise to differential thermal expansion and thermally induced curvature.

The effect of such heating on a vertical panel is illustrated in Figure 4. The panel shown in this figure is laterally supported near the top and "pinned" at the base. The thermally induced curvature leads to bearing against the outer flange of the column and this causes the panel to *attempt* to deflect outwards as a cantilever. Assuming that the lateral support at the top of the wall will *allow* such movement, the lateral deflections may be quite substantial (> 100mm) at the point of attachment. If the wall is held *rigidly* against the column, and if the column has substantial strength, large forces may develop at the connection and it is quite likely that the connection will fail. Such connection failures have been observed in several warehouse fires. The connection between wall panels

Figure 1. Types of steel framed buildings

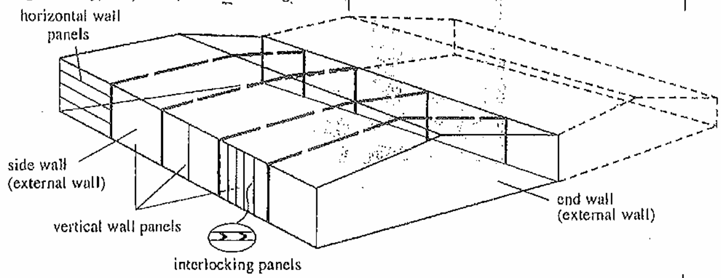


Figure 2. Support of vertical wall panels - wind forces

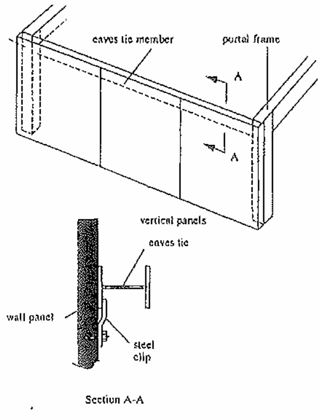
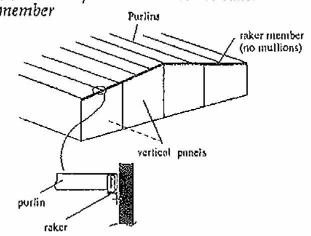


Figure 3. Various types of wall systems
(a) Vertical panels connected to raker member



(b) Horizontal and vertical panels with mullions and rafter

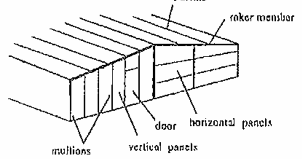
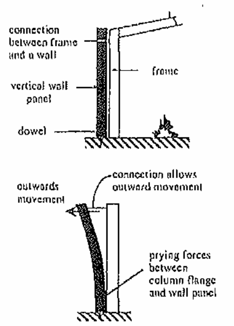


Figure 4. Forces at connection for vertical side wall panels



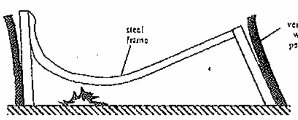
and supporting columns must not fail: and therefore should allow for substantial *relative* deflection between the panel and the column at the point of attachment.

The above discussion assumes that the bending strength of the supporting column is sufficiently large to offer substantial resistance to the outwardly deforming wall. This will not be the case where light columns are used (ie. columns having a bending capacity (ϕMs) 50kN-m), and in these situations no special allowance need be made for the relative displacement between panels and column.

2.1.2 Later Stages of Fire Growth

As the fire develops further, the walls will continue to deform outwards and the steel frame undergo substantial deformation. Advanced analysis [2] has shown that as high temperatures are achieved in the steel frame, it will take up the shape shown in Figure 5. Observations of steel frames after fires in single storey buildings have generally confirmed the correctness of this predicted shape.

Figure 5. Deformation of steel frame in later stages of heating



Provided the panels remain attached to the frame, the external walls will continue to provide adequate fire separation to an adjacent building - even though the panels are no longer vertical and small gaps may have developed between the panels. This was the finding of the AUBRCC study reported in reference [3], and it is on this basis, that the BCA continues to allow the non-fire protection of the roof or supporting columns in these situations.

Not only will the steel supporting structure get hot, but so will the connections between the panels and the supporting structure; and therefore an appropriate allowance must be made for the reduction in strength of the connection due to elevated temperature. For the purpose of preparing the standard details presented in this publication, it has been assumed that the maximum temperature of the steel connection is 750—800°C. Given the venting available through the roof of the building, in the event of a large fire, it is considered unlikely that this temperature will be exceeded for the connections. The temperatures of bolts attaching the panels to a supporting member, or between the supporting member and the frame, are likely to be lower than the above values due to the heat sink effect of the concrete and surrounding steel and the lower exposed surface area to mass ratio associated with bolted connections.

As will be noted from Figure 5, the columns on one side of the frame will slope outwards and the columns on the other side of the frame inwards. In the case of the columns sloping outwards, the connections between the frame and the side wall panels must hold the panels in this position throughout the latter stages of the fire. On the other side of the frame, the panels must be pulled inwards with the frame. This gives rise to the greatest force at the connections between the frame and the panels. This force must be sufficient to overcome the P-δ effects associated with the outwards movement of the wall plus the bending resistance of the wall at its base. This is illustrated in Figure 6. The outwards displacement of the wall is taken as one tenth of the height of the wall (see BCA Specification C1.11).

2.2 Design Details

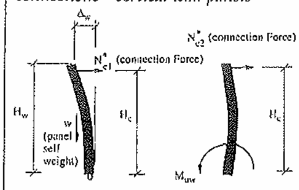
2.2.1 Introduction

Design details for the connection of vertically spanning side wall panels to the supporting frame are presented in Section 2.2.2 below, and were obtained by considering a range of building situations, and calculating the maximum deformations and forces associated with the connections between the supporting frame and the side wall panels. Due allowance has been made for the effect of elevated temperature on the capacity of the unprotected steel connections.

2.2.2 Details

The deformable column tie shown in Figure 7(a) can be used to connect the vertical panels back to the supporting columns subject to the following restrictions:

Figure 6. Derivation of force at connections - vertical wall panels



$$N^*_{c1} = \frac{\Delta_w L_{wc} w H_w}{2 H_c}$$

"P-δ" Effects

$$N^*_{c2} = \frac{M_{uw}}{H_c}$$

Overcoming base strength

Total Connection Force, $N^*_c = N^*_{c1} + N^*_{c2}$

H_c	the height from the base of the wall to the uppermost connection between panel and the supporting structure (m).
L_c	the distance between successive columns (m).
L_{wc}	the width of wall panels laterally supported by a connection to a panel (m).
M_{uw}	the ultimate moment capacity (excluding ϕ factor) at the base of a vertical panel including consideration of the base connection details but ignoring the effects of axial compression in the panel (kN-m per m), and the cracking moment.
N^*_c	the design load (axial tension) required to be developed at a connection to a panel (kN).
Δ_w	outwards displacement of the top of the wall (mm).
w	the weight of wall panel per unit area (kN/m²).

i. the height of the vertical panels must be $\leq$ 9m (Figure 8).

ii. the height of the column tie must be 2m or less from the top of the wall panel (Figure 8).

iii. the supporting columns must be spaced at $\leq$10m and a pair of deformable column ties utilised at each column location (Figure 9).

iv. the vertical panels must be each connected to the supporting columns or designed such that intermediate non-attached panels are effectively connected together. This will be achieved if the panels are large singular panels, or if they are interlocking. However, in the case of smaller singular panels, which are not of the interlocking type, the panels must be linked together (see Figure 9).

The deformable column tie shown in Figure 7(b) is appropriate for situations where the panel height is 7.5m and the supporting columns are spaced $\leq$ 7.5m. All other conditions specified in i - iv above must be satisfied.

If the columns adjacent to the panels have a bending capacity (ϕM_s) $\leq$ 50kN-m, then it

Figure 7(a) Deformable column tie (for 360UB or larger)

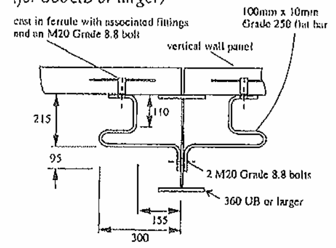


Figure 7(b) Deformable column tie (for 250UB or larger)

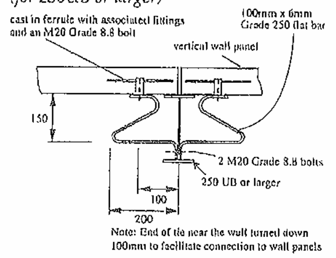


Figure 8. Geometric restrictions

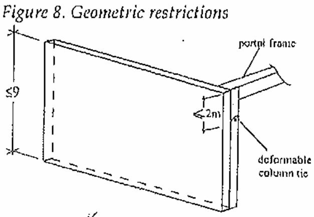
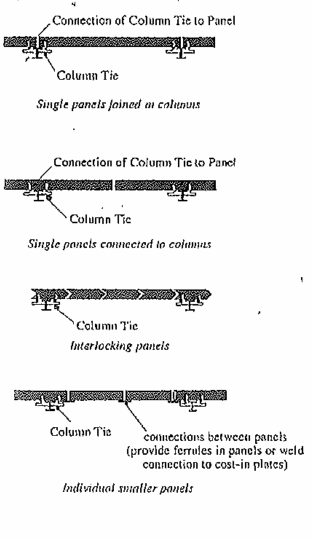


Figure 9. Support of various panels with deformable column ties (plan views)



is not necessary to connect the panels using deformable column ties. In this case, a more rigid form of connection is acceptable, provided the connection can achieve a tensile strength equal to that of the deformable column tie.

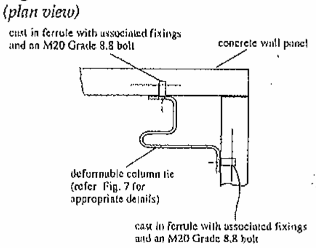
2.2.3 Side Wall to End Wall Connection

In many situations a column will not be present at the junction between the side and end walls - thus making it impossible to connect the side wall panels near the corner of the building back to a column. In this case the side wall panels should be restrained by connecting them to the end wall panels.

Due to the tendency of both side and end wall panels to bow outwards, any connection between side and end wall must be able to withstand extensive deformation. Adequate connection between side and end wall panels can be achieved by a single deformable column tie (see Figure 10) - the positioning and limitations associated with its use being those specified in Section 2.2.2 above.

3. Support of Vertical Panels - End Walls

Figure 10. Side wall to end wall connection (plan view)



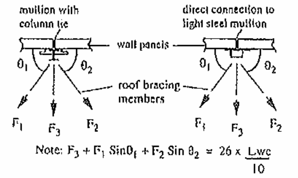
3.1 Background

As noted in the Section 1, it is possible that substantial mullions (ie. those having a bending capacity (ϕM_s) > 50kN-m) may be used as part of an end wall frame. In this case, the interaction of panels and mullions is exactly the same as described in Section 2.1 for side wall panels, and the same design details are applicable. In this case however, roof members must be provided within the first bay to give direct lateral load resistance to the mullions (see Figure 11).

If there are no mullions, or if the mullions are only relatively light (ie. having a bending capacity (ϕM_s) ≤ 50kN-m), and the panels are to be connected directly to the mullions, then it is not necessary to connect the panels using deformable column ties. In this case, a more rigid form of connection is acceptable, but the connections must have sufficient strength, at elevated temperatures, to overcome the P-δ effects and the bending resistance at the base of the wall. In addition, roof members must be provided within the first bay to give direct lateral load resistance to the mullions (Figure 11).

Alternatively, a raker member may be connected directly to the panel and to the roof purlins (see Figure 3(a)).

Figure 11. Restraint to mullions at end walls



3.2 Design Details

End Walls with Substantial Mullions.

In the case of an end wall framing system incorporating mullions having a bending capacity (ϕM_s) > 50kN-m, the design details given in Figure 7 and the restrictions of Section 2.2, are applicable.

Roof or bracing members within the first bay must be provided to give direct lateral support to the mullions. This may be considered to be achieved if:

(a) the capacities of the fasteners and members providing lateral support to the mullions are reduced to 10% of their ambient temperature values and

(b) the members and fasteners, with capacity reduced according to (a), are capable of resisting a tensile force of $26\times\frac{L_{WC}}{10}$ kN , where L_{WC} is the width of wall panels supported by a mullion in metres.

End Walls with Lightweight Mullions or Raker Member

In the case of end walls where there are no mullions, or where the mullions have a bending capacity (ϕM_s) ≤ 50kN-m, the design details given in Figure 12 are applicable subject to the following restrictions:

i. the height of the vertical panels must be ≤ 9m.

ii. the height of the connection must be 2m or less from the top of the wall panel.

iii. in the case of the detail given in Figure 12(a) the supporting mullions must be spaced at ≤ 10m and a pair of cleat plates utilised at each column location. In addition, the vertical panels must be each connected to the supporting columns or designed such that intermediate non-attached panels are effectively connected together. This will be achieved if the panels are large singular panels or if they are interlocking. However, in the case of smaller singular panels, which are not of the interlocking type, the panels must be linked together (see Figure 9).

Roof or bracing members within the first bay must be provided to give direct lateral support to the mullions. This may be considered to be achieved if:

(a) the capacities of the fasteners and members providing lateral support to the mullions are reduced to 10% of their ambient temperature values and

(b) the members and fasteners, with capacity reduced according to (a), are capable of resisting a tensile force of 26x $\frac{L_{WC}}{10}$ kN, where L_{WC} is the width of wall panels supported by a mullion in metres.

(c) in the case of the detail given in Figure 12(b), each purlin must be connected to the raker member and each panel must have at least two connections to the raker member unless the panels are of the interlocking type. The spacing of connections between the panels and raker member must not exceed 2m.

4. Support of Horizontal Panels

4.1 Background

In the case of horizontal panels, the panel is normally connected to the column with steel clips located close to the corners of each panel, as shown in Figure 13.

Under fire exposure, the panels will bend and this may result in the clips being opened as the panel pries against the column flange (see Figure 14). If the clips are not welded to the column flange, they may rotate, and fail to remain in contact with the flange.

As the fire further develops, the columns (with attached panels) along one of the side walls will lean outwards, and in this case, the connections at the corners of the panels must resist the "P-δ" forces arising from the self weight of the panels. Although these forces are relatively small, the use of non-welded clips in conjunction with sustained forces and high temperatures, may result in continued opening of the clips and eventual disengagement of the panels from the supporting structure.

4.2 Design Details

The connection detail shown in Figure 15 can be used to connect the horizontal panels to the supporting columns subject to the following restrictions:

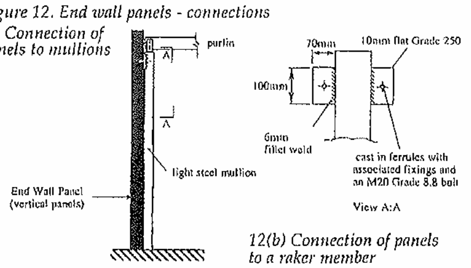
i. horizontal panels must be ≤ 10m in length

ii. the height of horizontal panels must be ≤ 2m.

iii. four connections are required per panel.

Figure 12. End wall panels - connections

(a) Connection of panels to mullions



12(b) Connection of panels to a raker member

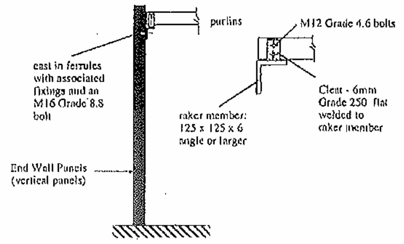


Figure 13. Connection details for horizontal wall panels

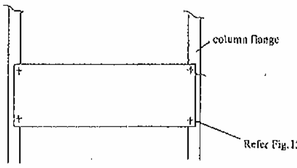


Figure 14. Bowing of horizontal wall panel

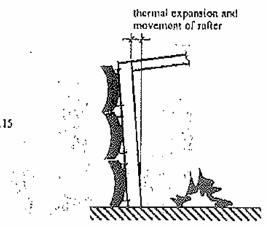
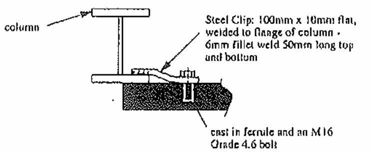


Figure 15. Support of horizontal wall panel



5. References

[1] Building Code of Australia, including Amendment No 7; Australian Building Codes Board, October 1994.

[2] Design of Single Storey Industrial Buildings for Fire Resistance, O'Meagher, A J, Bennetts, I D, Dayawansa, P H, and Thomas, I R, Journal of Australian Institute of Steel Construction, Vol 26, No 2, May 1992.

[3] Fire Protection of Steel Framing - AUBRCC Research Report AP25, O'Meagher, A J, Bennetts, I D, Dayawansa, P H, and Thomas I R, BHP Research - Melbourne Laboratories Report No. BHPRML/CM7/90/001, 1991.

For further information please contact:
BHP Research - Melbourne Laboratories
Phone 560 7066
or BHP Structural Steel Development Group
Phone 609 3778