Developments in Fire Engineering Design of Steel Framed Multi-Storey Buildings and Their Implementation:

Presentation to the Structures in Fire Forum
17 Sept 2012
By Associate Professor G Charles Clifton,
The University of Auckland
Scope of Presentation

Steel Building Behaviour in Severe Fires - Overview

Slab Panel Method
- Basis of design procedure
- Structural performance to be delivered
- Building structure characteristics and detailing requirements
- Background to procedure development
- Future research planned

Radiation Barrier Method

Verification Method for Fire Safety Design
- Structural performance
- Design of steel Structures

Performance of Composite Slabs in Earthquake
- Lessons from Christchurch 2010/2011
Steel Building Behaviour in Severe Fires

Brief overview
Multi-storey Building Response to Severe Fires

For steel buildings lot of data. Key points from this are:

• performance of the system is different to that of the individual members
• fire resistance of the system is typically higher than that of the individual members
• inelastic response will occur in fully developed fires
• detailing is important especially with unprotected beams
Behaviour of Steel Columns

- Gravity load pre-heat and wind load during fire
- During fire, columns heat up and expand
- Restraint of this expansion increases column compression load
- Effect on column depends on:
  - Individual column strength, stiffness
  - Restraint of surrounding structure
  - Temperature gradient
  - Redundancy

Stocky Column
- With Local Buckling at Top

Slender Columns
- With Member Buckling
Behaviour of Braces in Steel Systems

- Braces will be part of either an EBF or a CBF seismic-resisting system
- Their size is governed by seismic actions and the gravity actions on them prior to a fire will be very low
- Can sometimes be left unprotected
Behaviour of Steel Beams

- Gravity load pre-fire and wind load during fire
- During fire, temperature gradients cause beams to develop downwards sag
- During fire, increase in temperature causes beams to try and expand
- Two effects tend to minimise expansion effects on columns from beams but increase rotational demand on connections and impose tension forces on cooling down

Steel at 1100 ºC

Fire at 1200 ºC
Behaviour of Steel Frame Connections: 1

- During the heating phase, the connections are subject to negative rotation and minor to significant axial compression.
- During the cooling phase, there is some reversal of rotation and significant axial tension.
- The effects are much greater for unprotected beams.
- Connections with no inelastic reserve of strength in fire must be avoided.

Examples of connection failure during cooling down, from Cardington tests.

Fracture of flexible end plate.
Behaviour of Steel Frame Connections: 2

- The rotational requirements for ductile response in earthquake are also suitable for fire
- Reinforcement in the slab is important for simple connections
- Loss of vertical load carrying capacity must be avoided
- Failure of bolts or welds must be avoided
- Suitable connection details are in published NZ industry documents for earthquake design and performed very well in Christchurch
  - eg SCNZ Report 14

![Web side plate (WP)](image1)

![Flexible end plate (FE)](image2)
Behaviour of Floor Systems

- The critical direction for fire attack on a floor system is from underneath.

Floors must meet requirements for structural stability, integrity and insulation.

Many types of floor system have been shown to meet these requirements and have a high inelastic reserve of strength in severe fire conditions.

Not all floor systems are potentially capable of this behaviour.

Composite floors show good inelastic behaviour, but... Well detailed and built composite slab behaviour around supporting column. Full depth fracture must be suppressed.
Unprotected Steel Behaviour in Severe Fires

- Exposed steel elements may reach close to the temperature of the surrounding gases.
- Beams will sag downwards at an initially rapid rate, then a decreased and constant rate with time.
- Columns will undergo increased compression loading due to restrained thermal expansion.
- The extent of deformation depends on:
  - Is member shielded by an effective radiation barrier?
  - If directly exposed, what is expected maximum fire temperature and duration?
The Slab Panel Method
Under ambient temperature conditions:

- The beams support the floor slab
- One way action prevails
- Load path:
  slab $\rightarrow 2^0$ beams $\rightarrow$
  $1^0$ beams $\rightarrow$ columns

Under severe fire conditions:

- Unprotected secondary beams lose strength
- Two way action prevails (slab panel)
- Slab panel supports the beams
- Load path: slab panel $\rightarrow$ supporting beams $\rightarrow$ columns
- Slab panel axial forces are in in-plane equilibrium
Under severe fire conditions:

- Slab and secondary beams may undergo appreciable deformation
- Support beams and columns undergo minimal deformation
- Tensile membrane response may be activated
- Load-carrying capacity and integrity are preserved for full burnout
- Insulation is met for required period
Suppression of structural damage controlled by:

- Shielding linings (limited effectiveness)
- Sprinkler protection (extremely effective)

Effective compartmentation is maintained:

- Between floors
- Between firecells, same floor
Building Structure Characteristics Required for Implementation of Slab Panel Design Procedure

1. Floor slabs
   - concrete: structural grade, NWC or LWC
   - mesh/reinforcement: within slab panel, any grade over supports ≥ 15% uniform elongation
   - solid slabs, trapezoidal and clipped pan deck shapes

2. Steel beams
   - UB, WB, light steel joists, cellular beams

3. Columns
   - UC, WC require passive protection, can use CFSTs

4. Connections
   - must maintain integrity during heating and cooling down
   - connector failure (bolts or welds) to be suppressed
   - same detailing as required for earthquake; NZ standard practice

4. Overall building stability
   - no limitations on lateral load resisting systems
   - building stability not endangered by use of SPM
Detailing Requirements

(1) Floor slab

- Decking fastened to beams; typically composite

(2) Protection to slab panel edge support beams

- When specified, apply over full length
- Details given for application around connections to secondary beams

(3) Protection to columns

- Apply over full length

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Diagram:

- DH 12 trimmer
- DH 12 edge bars; see notes 3-7
- Interior support bars:
- (0.15Lx + 600 mm) for fixed support; note 10
- General reinforcement
- Slab panel 1

- DH 12 trimmer 600 mm
- DH 12 edge bars; see notes 3-7

- General reinforcement
- Deck trough bar (optional)

- Edge and trimmer bar reinforcement in slab panel 2 is not shown

- 600 mm for simple support; note 9
- Slab panel 2

- DH 12 edge bars, see notes 3-7
- Interior support bars:

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Passive Protection
Surrounds Base of Cleat in Contact with the Primary Beam
Steps to Implementing a Slab Panel Design

First design the floor and structural system for gravity and lateral loading conditions, then:

**Step 1:** Determine the size of the slab panel and location of the slab panel supports

**Step 2:** Determine which of the internal supports can carry negative moment

**Step 3:** Start with recommended reinforcement contents

**Step 4:** Input all variables and check capacity; increase as recommended in report

Fig. 1 Reflective Floor Plan for Application of Slab Panel Fire Engineering Design Procedure to a Concrete Slab on Profiled Steel Deck Supported on Primary and Secondary Beams
Moment/Tensile Membrane Resistance

This uses the modified Bailey model, ie:

\[ w^* = G + Q_C \]

from Loadings Standard

\[ w_u = (w_{yl\theta} - w_{yl\theta,ss}) + w_{yl\theta,ss} e \]

\[ w_u \geq w^* \text{ required} \]

where:

- \( w^* \) = fire emergency distributed load
- \( w_u \) = slab panel load carrying capacity
- \( w_{yl\theta} \) = yieldline load carrying capacity in fire
- \( w_{yl\theta,ss} \) = simply supported yieldline load carrying capacity in fire
- \( e \) = tensile membrane enhancement factor
  
  \[ e = fn \left( L_x, L_y, m_x, m_y, t_{eq}, t_o, h_{rc}, f_{yr,\theta}, E_{yr,\theta} \right) \]

- \( t_o, h_{rc} \) are slab thickness, deck rib height
- \( f_{yr,\theta}, E_{yr,\theta} \) are for reinforcement including secondary beams
Shear Resistance

This is additional to the Bailey model:

\[ w^* = G + Q_u \]

\[ v^* = w^*(L_x / 2) \]

\[ v_{u,\text{slab}} = \phi_{\text{fire}} v_c d_v \]

\[ \phi_{\text{fire}} = 0.89 \text{ from standard} \]

\[ v_c = \text{conc. slab shear capacity} \]

\[ d_v = \text{effective shear depth} \]

\[ V_{u,\theta,\text{sb}} = \text{shear capacity of secondary beam in fire} \]

\[ S_{\text{sb}} = \text{spacing of secondary beams} \]

\[ v^* \leq v_{u,\text{slab}} + \frac{V_{u,\theta,\text{sb}}}{S_{\text{sb}}} \text{ required} \]
Development Work Undertaken

- 18 stage experimental and analytical development programme undertaken
- Steps presented in following slides
- Covers from 1995 to 2012

- Demonstrated performance of large scale composite floor systems
- Showed systems with unprotected beams and protected columns have high fire resistance
Step 2: BRE Design Model and Test 2000

- Colin Bailey Tensile Membrane Model, UK BRE
- Large scale ambient temperature tests on lightly reinforced slabs to validate behaviour
• Generalised application of Bailey model for review
• HERA DCB No 60, February 2001
• Incorporating moment capacity of secondary beams
• General formula for yieldline determination
  includes support moment contribution
• Limits on application set by Bailey for:
  – integrity
  – maximum deflection
Step 4: FEM of Cardington Test Building 2002 published 2004

- Modelling of Cardington BRE large scale fire test
- Set of interlinked composite beams
- Interlinking required to obtain good agreement with experimental deflected shape
- Showed the two way nature of the floor system behaviour must be considered to replicate experimental behaviour

BRE large compartment test

- part of PhD research project
- details as shown opposite and below
- all slabs withstood 180 minutes ISO fire without failure: see next slide

<table>
<thead>
<tr>
<th>Slab</th>
<th>Thickness</th>
<th>Mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 661 flat slab</td>
<td>100mm</td>
<td>661 mesh</td>
</tr>
<tr>
<td>2 HD12 flat slab</td>
<td>100mm</td>
<td>HD12 bars</td>
</tr>
<tr>
<td>3 D147 flat slab</td>
<td>100mm</td>
<td>D147 mesh</td>
</tr>
<tr>
<td>4 Hi-bond slab</td>
<td>130 mm</td>
<td>D147 mesh</td>
</tr>
<tr>
<td>5 Traydec slab</td>
<td>130 mm</td>
<td>D147 mesh</td>
</tr>
<tr>
<td>6 Speedfloor slab</td>
<td>90 mm</td>
<td>661 mesh</td>
</tr>
</tbody>
</table>
Results of tests

D147 top surface crack pattern

Load ratio $\leq 1.0 \Rightarrow$ no tensile membrane enhancement required

Load ratio $> 1.0 \Rightarrow$ tensile membrane enhancement is required
• Incorporating results of furnace tests
• HERA DCB No 71, February 2003
• Improved determination of slab and reinforcement temperatures
• Revised reinforcement limits for integrity
• Relaxation of maximum deflection and limits on e
Step 7: Development and Validation of FE Model 2003

- 6 test slab panels modelled
- Best fit to mid-span deflection made for each case
- Accuracy of models also compared with:
  - reinforcement strains
  - edge deflections and rotations

Example shown for Speedfloor slab
Step 8: Determining the Influence of Deforming Supports on Slab Panel Behaviour 2004

FEM used to extend experimental testing to determine the influence of:

- effect of deformation in slab panel edge supports (no effect on capacity; increases panel midspan deformation, 65% contribution)
- horizontal axial restraint is significant, even at low levels (100kN/m stiffness)
- slabs of 4.15m x 3.15m, 8.3m x 6.3m and 8.3m x 3.15m analysed: 8.3m x 6.3m result shown below
Step 9: Confirming the SPM Assumption on Secondary Beam Contribution to Slab Panel Behaviour 2004/2005

FEM used to extend experimental testing to determine the influence of:

- contribution of the unprotected secondary beams: contribute to slab panel moment resistance as shown below

![Diagram showing the forces and components](image)

All steel tension forces are calculated for their design elevated temperatures
Step 10: Comparison of SPM Prediction with FEM for Real Floor System 2004/2005

- First analysis of a complete floor system
- 550m² 19 storey building built 1990
- Trapezoidal decking on secondary beams with central primary beam
- Floor divided into two slab panels
- This is the design example used in all editions of the procedure
Step 11: Distribution of Slab Panel Loads into Supporting Members for Strength Determination 2005

- Based on yieldline pattern
- Important is realistic to avoid support beam failure and slab panel plastic collapse (Abu)
- FEM modelling showed more realistic than ambient temperature design practice

<table>
<thead>
<tr>
<th></th>
<th>G+Q Hand calc.(HC)</th>
<th>ABAQUS (ABQ)</th>
<th>((ABQ-HC)/ABQ)*100</th>
<th>Fire - 44min SPM</th>
<th>ABAQUS</th>
<th>((ABQ-SPM)/ABQ)*100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column-1 (A-5)</td>
<td>64.8</td>
<td>43.5</td>
<td>-49.0%</td>
<td>55.0</td>
<td>71.8</td>
<td>23.4%</td>
</tr>
<tr>
<td>Column-2 (B-5)</td>
<td>159.9</td>
<td>180.2</td>
<td>11.3%</td>
<td>148.8</td>
<td>130.0</td>
<td>-14.5%</td>
</tr>
<tr>
<td>50% of Column A-4</td>
<td>18.9</td>
<td>29.6</td>
<td>36.1%</td>
<td>32.6</td>
<td>31.2</td>
<td>-4.5%</td>
</tr>
<tr>
<td>Total</td>
<td>243.6</td>
<td>253.3</td>
<td>3.8%</td>
<td>236.4</td>
<td>233.0</td>
<td>-1.5%</td>
</tr>
</tbody>
</table>
Slab panel central vertical downwards deflection versus time shows three stages of behaviour:

Stage 1: Decreasing rate of deflection with time due to thermal effects

Stage 2: Constant rate of deflection with time due to loss of yieldline capacity balanced by enhanced tensile membrane resistance

Stage 3: Increasing rate of deflection to fracture

- $\Delta_{\text{limit}} = \min (\Delta_1, \Delta_2) C_{\text{ISO}}$
- $C_{\text{ISO}} = 0.00743 \ t_{\text{eq}} + 0.7768$
- $\Delta_1, \Delta_2$ based on Bailey limits
Step 13: Third and Current Edition Published 2006

- Peer reviewed internationally
- Now used in most multi-storey composite steel floor fire engineered buildings in New Zealand
Example of SPM application to office building: 2007

**Note:**
1. These drawings must be read in conjunction with the fire report and structural fire report.
2. These drawings are intended to support the fire report. They do not show all of the required fire safety features specified in our fire report.
3. This drawing must not be considered to be a detailed construction drawing. Detailed design of the various elements illustrated is to be undertaken by others.
4. Unless instructed otherwise by the fire engineer, all fire separations must extend to the underside of the concrete slab or floor/roof cladding above with all gaps at the wall/concrete floor/junction appropriately fire and smoke stopped.
5. For beams or columns fully or partially enclosed in fire separations walls, integrity of the fire separation must be maintained regardless of the dispensations for passive fire protection that apply for structural stability (i.e. wall linings must extend around structure as required to comply with the specification for construction of the fire separation.)

- Cellular beams spanning parallel to numbered grids require transverse minimum slab reinforcing 147 mm2/m; bar spacing not more than 260 mm (not additional to conventional slab reinforcing)
Step 14: Incorporating Orthotropic Reinforcement Conditions into Tensile Membrane Model 2008/2009

- Undertaken by AP Tony Gillies, Lakehead University, Canada and graduate students
- Incorporates tensile membrane model updates from Bailey
- All applications are orthotropic due to temperature gradient effects even in regular slabs
• Correct orientation of tensile membrane fracture plane
  – tensile membrane fracture may be in Lx or Ly direction
  – whichever is the weaker

• Maintaining equilibrium at yieldline intersections
  – Steel across yield-lines cannot be above yield
Step 15a: Consideration of “double dipping” in regard to tension action in slab panel

- Can tension action in reinforcement and beams be used in yieldline moment and tensile membrane enhancement?
- Yes, until a full height fracture crack opens up along a yieldline

If \( R_{tsy} < R_{tsx} \) (long direction weaker):
- Final fracture not along yieldline
- No loss of yieldline moment capacity due to tensile membrane action

If \( R_{tsx} < R_{tsy} \) (short direction weaker):
- Final fracture along yieldline CD
- Loss of yieldline moment capacity near final collapse
- Beyond time to failure predicted from method
Step 16: Including Limitation Based on Compression Failure of Concrete Compression Ring 2010

- Avoidance of concrete compression failure in edge of slab
- Calculation of design width of concrete in compression
- Ensuring this is not also included in composite slab contribution to supporting beam
Step 17: Critical Review of Design Temperatures of Unprotected Secondary Beams within Slab Panel and SPM Deflection Limits 2011

Current 4\textsuperscript{th} year student project due for completion September 2011

Objectives:

1. Review temperatures used for unprotected steel beams in SPM 2006 against 6 recent large scale fire tests
2. Review relationship between fire gas temperature and steel beam temperature against same 6 tests
3. Review calculated deflections against test deflections
4. Make recommendations for changes to SPM 2006 criteria

Tests used:

1. Cardington Demonstration Furniture Test 1995
2. Cardington Corner Test 1995
3. Cardington Corner Test 2003
4. Mokrsko
5. FRACOF
6. COSSFIRE
## Step 17: Critical Review of Design

Temperatures of Unprotected Secondary Beams within Slab Panel and SPM Deflection Limits 2011

<table>
<thead>
<tr>
<th>Fire test</th>
<th>$\phi_{fire} w_u$</th>
<th>$w^*_test$</th>
<th>$w^*<em>test/\phi</em>{fire} w_u$</th>
<th>$\Delta_{limit}$</th>
<th>$\Delta_{test}$</th>
<th>$\Delta_{test}/\Delta_{limit}$</th>
<th>$t_{eq}$</th>
<th>Notes on $t_{eq}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cardington Furniture Test</td>
<td>7.09</td>
<td>4.94</td>
<td>0.7</td>
<td>726</td>
<td>642</td>
<td>0.88</td>
<td>54</td>
<td>Calculated from $t_{eq} = e_{i} k_{b} w_{f}$</td>
</tr>
<tr>
<td>Cardington Corner Test</td>
<td>6.47</td>
<td>4.94</td>
<td>0.76</td>
<td>754</td>
<td>388</td>
<td>0.51</td>
<td>62</td>
<td>Calculated from $t_{eq} = e_{i} k_{b} w_{f}$</td>
</tr>
<tr>
<td>Cardington 2003 Test</td>
<td>5.25</td>
<td>7.15</td>
<td>1.36</td>
<td>777</td>
<td>919</td>
<td>1.18</td>
<td>57</td>
<td>Calculated from $t_{eq} = e_{i} k_{b} w_{f}$</td>
</tr>
<tr>
<td>Mokrsko Test</td>
<td>7</td>
<td>6.6</td>
<td>0.94</td>
<td>864</td>
<td>892</td>
<td>1.03</td>
<td>65</td>
<td>Calculated from $t_{eq} = e_{i} k_{b} w_{f}$</td>
</tr>
<tr>
<td>FRACOF Test</td>
<td>19.55</td>
<td>6.89</td>
<td>0.35</td>
<td>750</td>
<td>460</td>
<td>0.61</td>
<td>120</td>
<td>Duration heating curve in furnace</td>
</tr>
<tr>
<td>COSSFIRE Test Option 1 (Note 1)</td>
<td>8.91</td>
<td>6.41</td>
<td>0.72</td>
<td>668</td>
<td>465</td>
<td>0.7</td>
<td>120</td>
<td>Duration heating curve in furnace</td>
</tr>
<tr>
<td>COSSFIRE Test Option 2 (Note 1)</td>
<td>4.19</td>
<td>6.41</td>
<td>1.53</td>
<td>668</td>
<td>465</td>
<td>0.7</td>
<td>120</td>
<td>Duration heating curve in furnace</td>
</tr>
</tbody>
</table>

Average value of 6 tests          | **0.81**         | **0.82**    |                             |                  |                |                               |        |                                            |

**Note 1:** The COSSFIRE test panel underwent a support failure of one short edge supporting beam. The first option is the SPM calculation on the basis of all support beams effective. The second option is the SPM calculation on the basis that one $L_x$ support beam is ineffective and therefore the slab panel length $L_y$ is doubled as that support becomes an effective centreline of a larger panel.
Step 18: Rewriting of SPM Software 2011 to 2012

- Much more user-friendly input/output
- Written in current version Visual Basic
- Data input screens include diagrams and explanatory text
- Currently in beta version
- QA over 2012/2013 summer
- Incorporates all 17 stages of development
- Demonstration to follow this presentation
Potential Future SPM Related Research
Strength and Stiffness of Slab Panel Edge Support Beams

- Following on from 2011 research into SPM
- Student has been appointed to start end 2012
- Supervisors AP Charles Clifton and Dr Tony Abu
- Status:
  - Slab panel support beams must have sufficient strength and stiffness to avoid a plastic collapse mechanism
  - What are the limits?
Contribution of Long Span Beams with Continuous Web Openings to Slab Panel Resistance

- These are becoming more common
- Student to be found
- Main Supervisor Charles Clifton? or Tony Abu?
- Status:
  - web contribution currently ignored
  - Is this accurate
• General determination following on from 2011 research
• Main Supervisor ??
• Student to be found
• Status:
  – Linus Lim in 2000 undertook PhD 6 slab panel tests and procedure verification
  – Repeat tests with fibres instead of general mesh
  – These used in conjunction with additional support reinforcement?
Determining the Adequacy of Slab Panel Detailing Provisions

- Main Supervisor ??
- Student to be found
- Determine by large scale experimental testing or modelling the adequacy of the current SPM detailing provisions
- Three large scale fire tests have recently supported the need for these with premature failures when details not included:
  - Mokrsko: slab pulled off slab panel edge support beam due to lack of edge and anchor bars around shear studs
  - Fracof: fracture of mesh where not adequately lapped within slab panel
  - VUT: shear failure at interior support where interior support bars too short and wrongly placed
Large Scale Fire Test

- Initial concept stage only
- Cost around NZD 3million
The Radiation Barrier Concept
The Radiation Barrier Concept

- A radiation barrier between unprotected steel and fully developed fire will keep the temperatures in the steel member below the limiting temperature as long as it remains in place.

- The presence of openings for lights and ventilation will not increase the steel temperature significantly.

- Once the failure condition for the barrier is met the barrier is gone and the steel from then on is exposed to the fire conditions as an unprotected member.

- A realistic model of the fire time-temperature conditions must be used in this determination.

- The steel members must remain below the limiting temperature as calculated by NZS 3404 (Steel structures standard).
• Two examples follow, one from the BHP Williams Street fire tests and the other from the Radiation Barrier project fire test no 3
• Williams St Test next slide
BHP fire test on actual enclosure. $T_{\text{lim}} = 540^\circ C$, $T_{\text{max}} = 535^\circ C$ in steel above ceiling

$T_{\text{fire}} > 1200 ^\circ C$ below ceiling which remained in place during fire
The aims of the experimental testing were to provide answers relating to the concept, namely:

- how effective are different radiation shields
- what is the influence of openings in ceilings
- develop failure criteria for standard and fire resistant gypsum plasterboard
- determine mode of failure of barriers
- accuracy of parametric fire models

MEFE Thesis Report Produced:
Brown, N.C. *Steelwork Partially Protected from Post-Flashover Fires in Gib® Board Lined Compartments*: ME (Fire) Thesis, Civil Engineering Department, University of Canterbury, 2005
Test Set-Up

- Three tests were undertaken
- Purpose built enclosure
Scope of Tests

Three tests undertaken

• Test 1: Fire Hazard Category 1 fire load, Gib® Standard ceiling, 13 mm thick
  – Fire load representative of apartments, 400 MJ/m$^2$ floor area; ceiling likely to fail during fire

• Test 2: Fire Hazard Category 2 fire load, Gib® Fyreline ceiling, 13 mm thick
  – Fire load 800 MJ/m$^2$ floor area, representative of offices; ceiling likely to fail

• Test 3: Fire Hazard Category 1 fire load, Gib® Fyreline ceiling, 13 mm thick
  – Fire load 400 MJ/m$^2$ floor area; ceiling expected to remain in place
Key Results of Tests

- Test 1: ceiling failed rapidly during full development of fire. Temperature on unexposed side at failure around 135ºC

- Test 2: ceiling failed during full development of fire. Temperature on unexposed side at failure around 275ºC

- Test 3: ceiling remained in place until well into cooling down period, when localised failure occurred. Temperature in ceiling void not above 150ºC
Development of Fire to Flashover: Test 3

Fire and Steel Temperatures from Test No 3

- Bot flange Bm A
- Bot flange Bm B
- Bot flange Bm A No 2
- Bot flange Bm B No 2
- Ave fire
- BF Temps FaST Mod Eur
- FaST Mod Euro Curve
Test Results vs Radiation Barrier Concepts

- Interim failure temperatures for the unexposed side of the Gib (gypsum) Board ceiling and walls have been determined.

- The presence of openings is not detrimental to the barrier.

- The design concept of barrier 100% in place until failure then not effective is realistic.

- The Modified Eurocode parametric fire curve is a reasonable prediction of the fire time temperature conditions for low fire load, high ventilation enclosures.
Gib Board Failure Temperatures

- Failure temperatures on unexposed face from research project
- For ceiling (below beams)
  - Standard Gib (gypsum) board 120°C
  - Fire resistant Gib (gypsum) board 250°C
- For walls (protecting columns)
  - Standard Gib (gypsum) board 200°C
  - Fire resistant Gib (gypsum) board 400°C
FaST: Fire and Steel Temperatures

Design Aid for Fire Engineering Design of Steel Structures. It provides the following capabilities:

- calculation of the Standard Fire time temperature conditions and two parametric curves
- calculate steel temperatures exposed to these fires
- calculate steel temperatures protected by a radiation barrier
How to Obtain and Use

- HERA Report R4-127 is commentary and use manual
- obtain at no cost from HERA website
  - www.hera.org.nz
- need to install Java Virtual Machine before can download
- Follow the steps in section 1.3 of the User Manual and Commentary; but
- Use Gib® failure temps from above slide
Application of FaST to a Radiation Barrier

- This uses a two stage calculation to get the heat from the fire into the steel, thus:
Example of Application

The result of FaST applied to the test 3 enclosure is as shown.

Fire and Steel Temperatures from Test No 3

- Bot flange Bm A
- Bot flange Bm B
- Bot flange Bm A No2
- Bot flange Bm B No 2
- Ave fire
- BF Temps FaST Mod Eur
- FaST Mod Euro Curve
Design Application of Method

1. Determine the firecell size, openings, thermal inertia value and fire loading

2. Determine the limiting temperature, $T_l$, for the steel members protected by the radiation barrier

3. Calculate the maximum temperature reached in the steel member behind the barrier, $T_{\text{max}}$

4. If $T_{\text{max}} \leq T_l$, the barrier is satisfactory. If not, increase barrier or use another method of insulation to the steel member
Advantages of Radiation Barrier Method

- Avoids need for additional passive fire protection to the structural steel
- Makes maximum use of linings
- Modelling of structural fire severity in accordance with C/VM2
  - as covered in later slides
- Minimises the damage to the structural system from fully developed fire
What Happens if the Fire is More Severe on the Barrier than the Design Fire?

• Barrier may fail when temperatures are high enough to take the steel members above the limiting temperature
• Very unlikely for columns, more likely for beams
• Result will be permanent local beam and floor slab distortion and need for local replacement of members
• Inelastic reserve of composite floor system will keep the floor functioning as an effective fire separation (ductile detailing used)
Verification Method
C/VM2

Introduced in April 2012
Mandatory for FED from April 2013
Expected to be used in majority of multi-storey buildings (low rise and above)
Is a new requirement
Title: Framework for Fire Safety Design
Scope: Fire engineering design of buildings
Objective:
1. Protect occupants from effects of fire
2. Protect other property (neighbour)
3. Protect fire fighting personal in rescue and fire fighting

The third objective is new to the NZBC in 2012 and the first time covered in the Compliance Documents.
Target users are professional fire engineers.
C/VM2 requires FED for 10 fire design scenarios
• **BE** = blocked exit
• **UT, CS, SF** = different types of developing fire for life safety occupants in enclosure of origin
• **HS, VS, IS** = horizontal, vertical, internal spread of fire
• **FO** = firefighting operations (principal scenario for structural robustness)
• **CF** = challenging fires (high growth or high toxicity)
• **RC** = robustness check on fire safety features (failure to operate scenarios)

Rest of slides focus only on **FO**
NZ Building Code Requirements for FO

These are the mandatory requirements. Paraphrased they are to:

- **Protect Other Property (Neighbour)**
  - Limit radiation flux at boundary
  - Limits on opening size
  - External openings to be stable

- **Facilitate Fire Fighting and Rescue**
  - Control fire fighter tenability conditions on arrival to firecell in unsprinklered firecells
  - Requirements for access
  - Structural systems required for firefighter safety must be stable during and after fire
FO Scenario Building Stability
Requirements from C/VM2

For buildings with escape height > 10m
a) Safe paths to resist burnout
b) Structure supporting floors to resist burnout

For buildings with escape height ≤ 10m
a) Safe paths can access all floors for 60 mins from ignition or burnout whichever less
b) Floor systems to resist collapse for at least 30 mins from ignition
c) From ignition to full development to be calculated (or taken as 10 mins)
1. Use a time equivalent formula and ensure \( FRR \geq t_e \)

2. Use a parametric time versus gas time temperature formula to generate gas time – temperature conditions for input into a structural response model

3. Construct a Heat Release Rate versus time design option then generate gas time – temperature conditions for input into a structural response model

Elaborating now on 1 and 2 and their application to steel framed buildings
Option 1: Time Equivalence Formula

\[ t_e = e_{f,\text{mod}} k_b k_m w_f \]

- \( e_{f,\text{mod}} \) = fire load energy density (MJ/m\(^2\) floor area) modified
- \( k_b \) = thermal inertia of firecell factor
- \( k_m \) = thermal inertia of firecell factor
- \( w_f \) = ventilation factor

and

\[ \text{FRR}_{\text{element}} \geq t_e \text{ required} \]
Fire Load Energy Density, $e_f$

- Table of activities given for each case for three design values; some examples
- Design FLED = 400 MJ/m$^2$ specified area
  - sleeping spaces, low fire hazard materials
- Design FLED = 800 MJ/m$^2$ specified area
  - offices, non bulk display of goods, some retail
- Design FLED = 1200 MJ/m$^2$ specified area
  - bulk storage of materials up to 3m high incl. libraries
  - plant and boiler rooms
## Modification to $e_f$

<table>
<thead>
<tr>
<th>Condition</th>
<th>Sprinklered firecell</th>
<th>Unsprinklered firecell</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elements in structures that cannot develop dependable deformation in post flashover fire \textsuperscript{Note1}</td>
<td>1.00</td>
<td>1.25</td>
</tr>
<tr>
<td>Elements that would cause disproportionate collapse</td>
<td>1.00</td>
<td>1.25</td>
</tr>
<tr>
<td>All other structural and non structural elements \textsuperscript{Note1}</td>
<td>0.5</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Note 1: Must be established by rational analysis + experimental testing

Composite slab on steel beams given in C/VM2 as the only system deemed to comply
Table 2.4 Conversion factor $k_b$ for various lining materials

<table>
<thead>
<tr>
<th>Typical values for $k_{pc} \sqrt{J/m^2s^{0.5}k}$</th>
<th>Construction materials</th>
<th>$k_b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>400</td>
<td>Very light highly insulating materials</td>
<td>0.10</td>
</tr>
<tr>
<td>700</td>
<td>Plasterboard ceilings and walls, timber floors</td>
<td>0.09</td>
</tr>
<tr>
<td>1100</td>
<td>Light weight concrete ceilings and floors</td>
<td>0.08</td>
</tr>
<tr>
<td>1700</td>
<td>Normal weight concrete ceilings and floors</td>
<td>0.065</td>
</tr>
<tr>
<td>&gt;2500</td>
<td>Thin sheet steel roof and wall systems</td>
<td>0.04</td>
</tr>
</tbody>
</table>

**NOTE:**

$k =$ thermal conductivity (W/m K)
$
\rho =$ density (kg/m$^3$)
$c =$ specific heat (J/kg K)
$k_m$ factor

$k_m = 1.0$ for reinforced concrete, timber or design procedure involving mix of unprotected and protected steel, eg as in SPM or RBM.

$k_m = \text{function of } A_v, h_{eq} \text{ and } A_t \text{ for individual unprotected steel members (rapid temperature rise in natural fires compared with in Standard Fire Test)}$
$w_f = \text{ventilation factor}$

$$w_f = \left( \frac{6.0}{H} \right)^{0.3} \left[ 0.62 + \frac{90(0.4 - \alpha_v)^4}{1 + b_v \alpha_h} \right] > 0.5$$

$A_f = \text{floor area of firecell}$
$A_v = \text{area of vertical openings in floor}$
$A_h = \text{area of horizontal openings in roof}$
$H = \text{inside clear height of the firecell}$
Option 2: Parametric Curve with Limiting Temperature

- Use either Eurocode or Modified Eurocode to get gas time temperature conditions, \( \theta_g \), then
- Get steel temps, \( \theta_s \), from gas temps; for unprotected downstand beams:
  - \( \theta_{\text{bottom flange, web}} = 0.95 \theta_g \)
  - \( \theta_{\text{top flange}} = 1.0 \theta_g - 150^\circ\text{C} \)
- Compare \( \theta_s \) with limiting steel temperature, \( T_l \)
  - if \( \theta_s \leq T_l \) design is OK
  - otherwise need to increase radiation barrier thickness as most effective approach
The Limiting Steel Temperature

A concept of limiting temperature developed based on individual member in Standard Test

- Lower bound determination fire resistance beyond this limit individual member loads redistributed due to continuity and redundancy

- Continuity
  - Simple end connection moment capacity

- Columns have upper limit on the calculated limiting temperature based on forces generated by the restraint of thermal expansion
  - Upper limit based on if column can develop plasticity

Example of beams heated to limiting steel temperature; BHP fire test.

\[ T_{\text{lim}} = 540^\circ\text{C}, \quad T_{\text{max}} = 535^\circ\text{C} \]
Carparks: Requirements and Options for Structural Steel

- \( e_f = 400 \, \text{MJ/m}^2 \) for actual carparking area (approx 50% of total floor area typically)
- structural fire severity determined floor by floor
- ventilation from external walls \( (A_v) \) and ramp to floor above \( (A_h) \)
- use either \( t_e \) and FRR or limiting temperature

For steel:
- use Slab Panel for floor system; area above 11 cars for closed; 5 cars open; all beams unprotected
- cols 20 to 30 mins FRR
Office and Retail Steel Framed Buildings With Composite Floors

- use SPM for floor system; most secondary beams unprotected
- $t_e$ calculated for $e_f$ with no modification
- protect columns to meet $t_e$ value; all bare steel columns require passive protection
- now standard approach in NZ
Apartments and Hotels

- steel behind acoustic linings; use as radiation barriers
- apply Radiation Barrier Method to each firecell
- firecell is apartment or hotel room
- vary calculated ventilation by $\pm$ 20%
- use Eurocode parametric curve for apartments; Modified Eurocode for hotel rooms
- detail floors for ductile response using SPM detailing
- all steel members typically unprotected
Performance of Composite Floor Systems in Severe Earthquake
Christchurch – between the sea and the hills: NZ’s second largest city
22 February Earthquake – Intensity of Shaking and Duration

Figure 1 NZS 1170.5 Spectra and Largest Horizontal Direction Recorded from the CBD Strong Motion Records

Notes:
1. The long dotted black line is the ULS design spectrum for normal importance buildings for the soft soil type, Class D, generally considered in the CBD
2. The short dotted black line is the Maximum Considered Event design spectrum for normal importance buildings for Class D soil in the CBD
3. The solid thick black line is the average from the 4 recording stations all of which are within 1km of the CBD and in similar ground conditions
Impact on CBD

Within the CBD zone:
• Over 1500 buildings before the events
• After all demolition completed around 10 to 15 will remain (quote from head of Canterbury Earthquake Recovery Authority for demolition)
The Earthquake Sequence: Impact on Christchurch CBD

Magnitude and Intensity of damaging events to date:

4 Sept 2010: M 7.1, MM 7, ≈ 0.7 x design*
26 Dec 2010: M 5.5?, MM 7 to 8
22 Feb 2011: M 6.3, MM 9 to 10, ≈ 1.8 x design*
6 June 2011: M 5.3?, MM 7 to 8
13 June, 2011: M 5.4?, MM 7 to 8
13 June 2011: M 6.3, MM 8 to 9, ≈ 0.9 x design*+
23 December 2011: M 5.5, MM 6 to 7, ≈ 0.6 x design*
25 May 2012: M 5.2, MM 5 to 6, ≈ 0.5 x design*

* design* = design for ultimate limit state to current seismic loading standard

Cumulative effect ≡ maximum considered event
Performance requirements of modern buildings in this level of event (>DLE)

For normal importance buildings to conventional ductile design, they:

- Shall remain standing under DLE, should also under MCE
- Structural and non structural damage will occur
- Building will probably require replacement
Range of Buildings Affected

- Ages from 1985 to 2010
- Number relatively low compared with precast concrete up to 2000; significant increase since then
- Range in height from 3 to 22 storeys
- Systems used;
  - Eccentrically braced frames (63%)
  - Moment resisting frames (50%)
  - Shear walls (13%)
  (some mixed systems hence > 100%)
  - 75% composite floors
  - 25% precast concrete + topping floors
Case Study: Club Tower

12 storey mixed EBF and MRF, composite floors, torsionally irregular

- Building has effectively self centred:
  - 45;35 mm out of plumb top; within construction tolerances
  - 0.14% maximum residual deflection

- Minimum damage
  - Lift guide rail realignment required: this has cost approx $250k
  - No other structural or non structural repair or replacement needed
  - Building now fully occupied including CERA and CCC
  - The only (normal importance) high-rise building in Christchurch now in use
1. Good management and technical robustness

- Capacity design procedure accounts for whole system performance
- Connection designs comprehensive and conservative
  - Includes minimum forces on connections and splices
- Continuous columns required in gravity and seismic-resisting systems
  - Assists with lateral stiffness and self-centering
Reasons for Good Performance

2. Properties and quality of steel and steel construction

- Steel has clearly defined yield point and only becomes inelastic under relatively high accelerations compared with concrete
- Steels used in New Zealand generally have good mechanical properties
  - Continuous cast, controlled rolled
  - High ductility and charpy impact properties
  - Highly consistent yield and tensile strengths
  - But not all steel met these criteria;
- Steel buildings generally well designed, detailed and constructed
  - Not much independent inspection so industry must police the standards: this has worked well for fabrication quality; not so well for material supply
3. Excellent performance of composite floors
   - Most robust of the floor systems in earthquake
   - More on this in next slide

4. Good luck
   - Capacity design procedure has accounted for whole system performance even with extra strengths from composite floor slabs that were not expressly accounted for in design and variation in steel strengths; therefore
     - Overall system behaviour still as expected, but
     - Inelastic demand lower than expected and
     - Damage threshold higher than expected
Excellent Performance of Composite Floors

They have demonstrated:

- Excellent diaphragm action
- Excellent interconnection with frames
- Elastic out of plane resistance (doubles strength and stiffness) which has self-centred conventional EBF/MRF systems
- Minor crack repair required only to some Pacific Tower floors
Strength and Stiffness: Actual versus Predicted

- Steel buildings typically 2 to 3 times stronger and stiffer than the models predicted: why: we are working on reasons – slab, SSI
- This determined from extent of observed response versus predicted response from model
- Most steel buildings effectively self-centred without need for specific devices to ensure this

HSBC Tower:
- Open plan office building
- Design drift 1.3% under DLE
- Actual drift ≈ 1% under 1.8 DLE
- Ratio of stiffness real/model = 2.3

Source: measurement of scuff marks on stairs; details from Design Engineer
Use of VULCAN to Model Slab Strength and Stiffness in Earthquake

- EBF behaviour forces slab out of plane
- Slab resists this generating extra resistance
Yield Line Model

- Linear spring model used
- Slab Panel Method used to find force at ultimate failure
- Half slab depth assumed for ultimate strain
Use of VULCAN to Model Slab Strength and Stiffness in Earthquake

- EBF behaviour forces slab out of plane
- Slab resists this generating extra resistance
- VULCAN being used to determine magnitude of this effect; research just started
Some First Results

Slab stiffness 20 – 30 kN/mm

Active Link
References

These are principally for the SPM slides


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