Steel Construction New Zealand

Slab Panel Method Workshop

An afternoon seminar for Design Engineers and Regulatory Authorities

Auckland 2nd September 2014
Christchurch 9th September 2014

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About the Presenters

Dr Charles Clifton,
University of Auckland

Charles has specialised in structural steel and composite engineering since joining the University of Auckland in 2008. This followed a productive period since 1983 as Senior Structural Engineer at the Heavy Engineering Research Association, where he conducted research in structural steel, composite construction, fire engineering and durability. He also made considerable contributions to the introduction of new and revised standards, developed widely used design guides and was actively involved in professional development. A long and productive collaboration with the University of Auckland saw many innovations researched, developed and adopted by the profession, and also saw the award of his PhD in 2005.

Charles is a Fellow of the Institute of Professional Engineers New Zealand and of the National Society for Earthquake Engineering. He is currently active in a range of research projects involving the development of low-damage seismic solutions, performance of composite steel floors in severe fires, and floor and frame solutions using light gauge steel members and components.

Dr Anthony Abu,
University of Canterbury

Dr. Anthony Abu is the New Zealand Fire Service Commission Lecturer in Fire Engineering at the University of Canterbury. Tony obtained his Bachelor's degree in Civil Engineering from Eastern Mediterranean University, North Cyprus and then completed his PhD in Structural Fire Engineering at the University of Sheffield, UK, on the behaviour of composite floor slabs in fire.

He has been involved in the implementation of the structural fire engineering Eurocodes in the UK and also worked on a number of structural, and structural fire engineering projects, including a number of sports stadia, office complexes and airports, during a brief period with Buro Happold Engineers Ltd. UK.

Development of the Slab Panel Method

By G Charles Clifton,
University of Auckland
and
Anthony Abu
University of Canterbury

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Scope of Presentation: SPM Development

Basis of design procedure

Structural performance to be delivered

Building structure characteristics and detailing requirements

Background to procedure development

Future research planned

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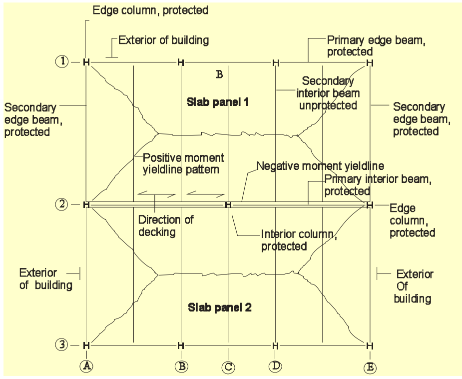
Basis of Design Procedure

Under ambient temperature conditions:

- The beams support the floor slab
- One way action prevails
- Load path:
  slab → 2^0 beams →
  1^0 beams → 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 → supporting beams → columns
- Slab panel axial forces are in in-plane equilibrium

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Structural Performance to be Delivered by the Procedure - 1 of 2

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 calculated t_e or specified FRR
- Insulation is met for required period

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Structural Performance to be Delivered by the Procedure - 2 of 2

Suppression of structural damage controlled by:

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

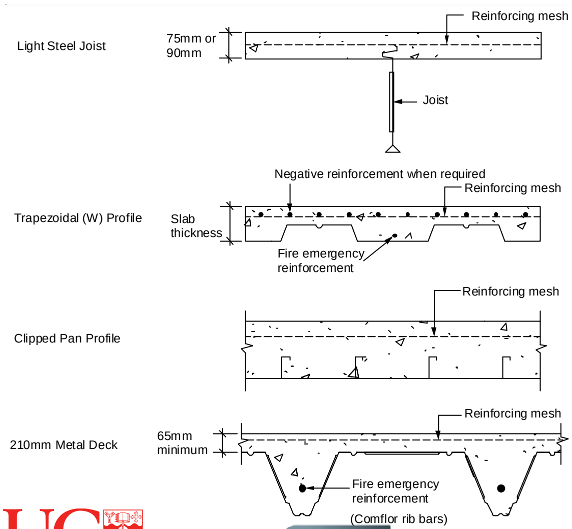
Effective compartmentation is maintained:

- 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



Building Structure Characteristics Required for Implementation of Slab Panel Design Procedure

(2) Steel beams
- UB, WB, light steel joists, cellular beams

(3) Columns
- UC, WC require passive protection in many applications, can use CFSTs
- Columns in car parking buildings typically don't require passive protection

(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

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Detailing Requirements

(1) Floor slab
- Decking fastened to beams; typically composite
- Slab tied to edge beams
- Shear failure at supports suppressed by shear reinforcement

(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 when needed
- Apply over full length

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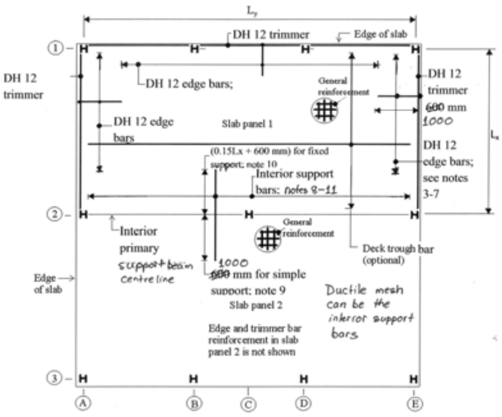
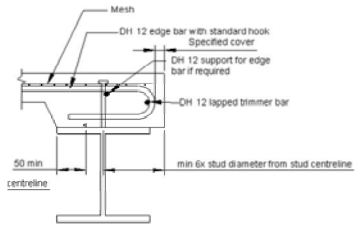
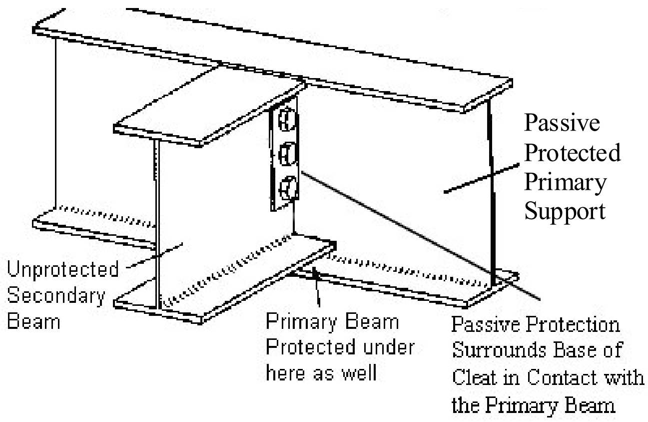
Detailing Requirements





Detailing Requirements



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

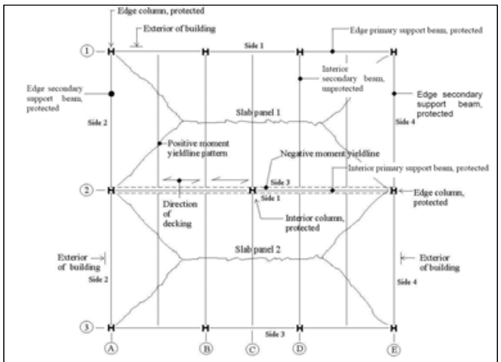


Fig. 1 Reflected Floor Plan for Application of Slab Panel Fire Engineering Design Procedure to a Concrete Slab on Profiled Steel Deck Supported on Primary and Secondary Beams

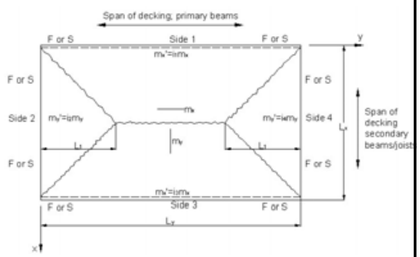
Moment / Tensile Membrane Resistance

This uses the modified Bailey model, ie:

$w^* = G + Q_C$ from Loadings Standard

$w_u = \left(w_{yl\theta} - w_{yl\theta,ss}\right) + w_{yl\theta,ss}e$

$w_u \geq w^*$ 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

= fn (L_x, L_y, m_x, m_y, t_e, t_o, h_{rc} $f_{yr,\theta}$, $E_{yr,\theta}$)

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:

$w^* = G + Q_u$

$$v^* = w^*(L_x / 2)$$

$$v_{u,slab} = \phi_{fire} v_c d_v$$

ϕ_{fire} = 0.89 from standard

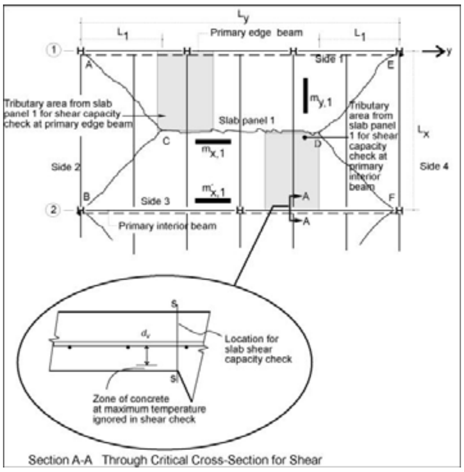
v_c = conc. slab shear capacity

d_v = effective shear depth

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

S_{sb} = spacing of secondary beams

$$v^* \le v_{u,slab} + \frac{V_{u,\theta,sb}}{S_{sb}} \; required$$



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Development Work Undertaken

- 22 stage experimental and analytical development programme undertaken
- Steps presented in following slides
- Covers from 1995 to 2014

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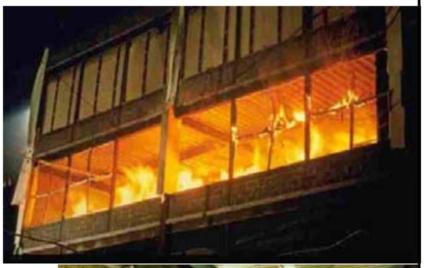
Step 1: Cardington Fire Tests 1995/1996 (and 2003)

- Demonstrated performance of large scale composite floor systems
- Showed systems with unprotected beams and protected columns have high fire resistance



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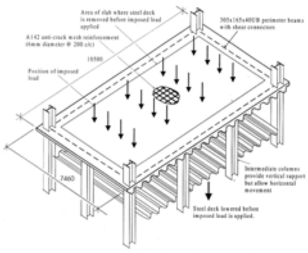
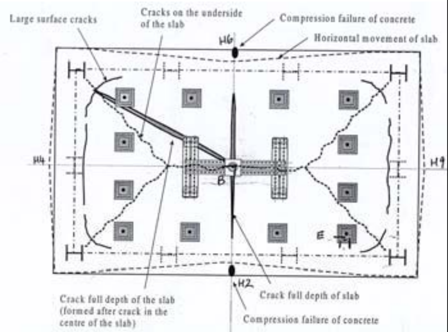
Step 2: BRE Design Model and Test 2000

- Colin Bailey Tensile Membrane Model, UK BRE
- Large scale ambient temperature tests on lightly reinforced slabs to validate behaviour





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Step 3: First Edition of SPM 2001

- 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

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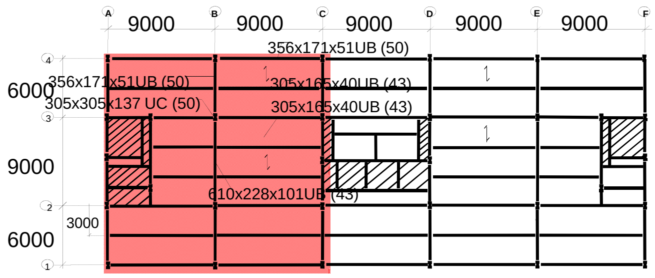
Step 4: FEM of Cardington Test Building 2002 published 2004

- 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

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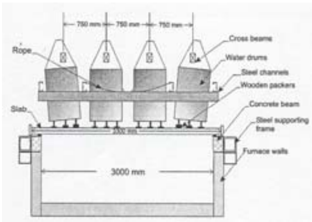
Step 5: Furnace Testing of Six Slab Panels 2001/2002

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

	Slab	Thickness	Mesh
1	661 flat slab	100mm	661 mesh
2	HD12 flat slab	100mm	HD12 bars
3	D147 flat slab	100mm	D147 mesh
4	Hi-bond slab	130 mm	D147 mesh
5	Traydec slab	130 mm	D147 mesh
6	Speedfloor slab	90 mm	661 mesh



Results of tests

D147 top surface crack pattern

	Slab	Applied load, w (kPa)	Ambient temperature		At 3 hours in the ISO fire		
			$W_{u,o}$ (kPa)	Load ratio, r_{load}	Max. Steel Temp. (°C)	$W_{u,f}$(kPa)	Load ratio, $r_{load,f}$
1	661 Flat slab	5.40	20.0	0.270	683	***2.40***	***2.25***
2	HD12 Flat slab	5.40	28.2	0.191	688	6.49	0.83
3	D147 Flat slab	5.40	13.3	0.406	703	***1.47***	***3.67***
4	Hibond slab	5.52	70.2	0.079	672	***1.09***	***5.06***
5	Traydec slab	6.12	75.0	0.082	339	8.57	0.71
6	Speedfloor	5.16	55.1	0.094	623	***2.02***	***2.55***

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

Load ratio > 1.0 ⇨ tensile membrane enhancement is required

Step 6: Second Edition of SPM 2002/2003

- 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*

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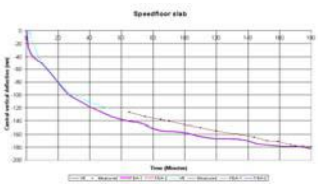
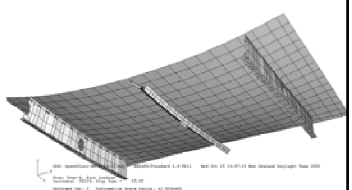
Step 7: Development and Validation of FE Model 2003

- 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





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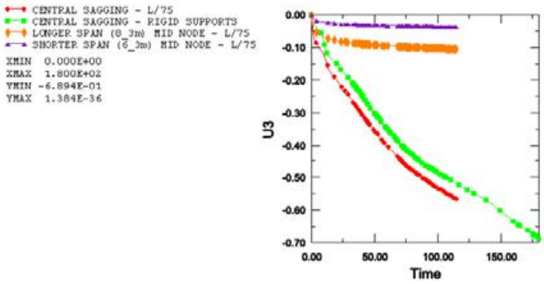
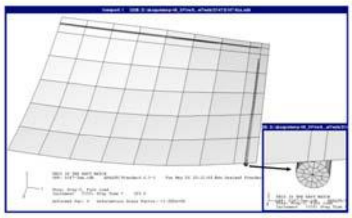
Step 8: Determining the Influence of Deforming Supports on Slab Panel Behaviour 2004

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





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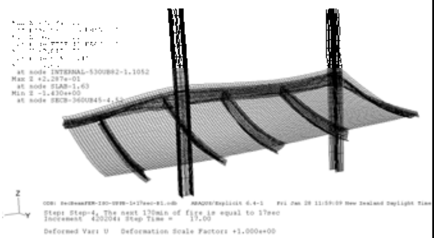
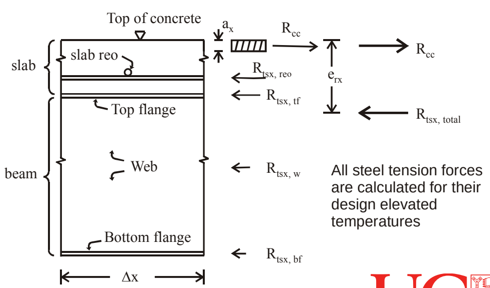
Step 9: Confirming the SPM Assumption on Secondary Beam Contribution to Slab Panel Behaviour 2004/2005

FEM used to extend experimental testing to determine the contribution of the unprotected secondary beams: contribute to slab panel moment resistance as shown below





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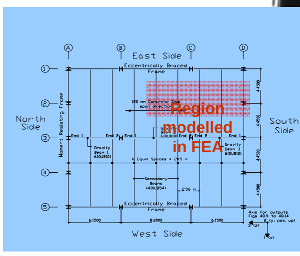
Step 10: Comparison of SPM Prediction with FEM for Real Floor System 2004/2005

- First analysis of a complete floor system
- 550m[2] 19 storey building built 1990
- Trapezoidal decking on secondary beams with central primary beam
- Floor divided into two slab panels
- This design example has been given in each edition of the procedure to keep a benchmark on the impacts of development of the model



Reflected Floor Plan

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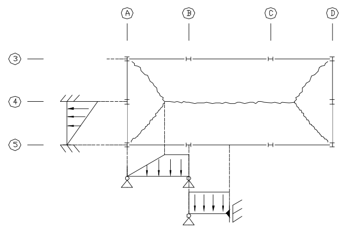
Step 11: Distribution of Slab Panel Loads into Supporting Members for Strength Determination 2005

- Based on yieldline pattern but with modifications from 2013 study: see application slides for changes
- This loading must be sufficient to avoid support beam failure and subsequent slab panel plastic collapse (Abu)
- FEM modelling showed that the two way deformation pattern is more realistic than ambient temperature design practice



	G+Q			Fire - 44min		
	Hand calc.(HC)	ABAQUS (ABQ)	((ABQ-HC)/ABQ)*100	SPM	ABAQUS	((ABQ-SPM)/ABQ)*100
Column-1 (A-5)	64.8	43.5	-49.0%	55.0	71.8	23.4%
Column-2 (B-5)	159.9	180.2	11.3%	148.8	130.0	-14.5%
50% of Column A-4	18.9	29.6	36.1%	32.6	31.2	-4.5%
Total	243.6	253.3	3.8%	236.4	233.0	-1.5%

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Step 12: Including Length of Structural Fire Severity on Limiting Deflection 2005/2006

Slab panel central vertical downwards deflection versus time shows three stages of behaviour in fire:

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. Some surface cracks in slab due to loss of moisture from concrete

Stage 3: Increasing rate of deflection with full depth cracks(s) forming and ultimately fracture of reinforcement crossing the crack(s)

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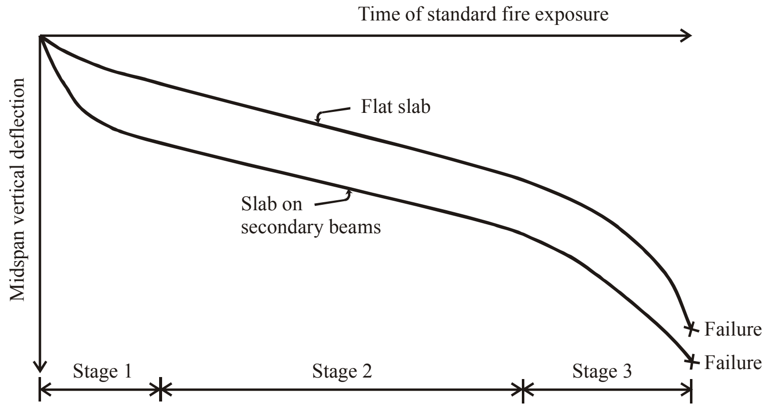
Step 12: Including Length of Structural Fire Severity on Limiting Deflection 2005/2006



SPM gives design capacity towards end of stage 2 behaviour; included through the C_{ISO} factor

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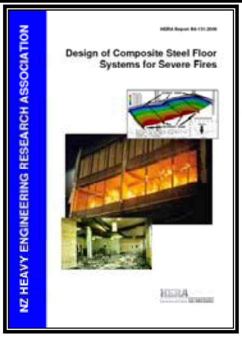
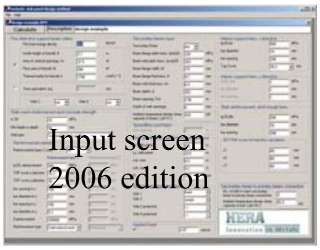
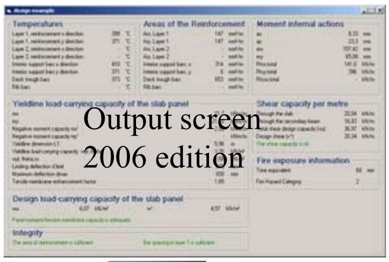
Step 13: Third Edition Published 2006

- Peer reviewed internationally
- Now used in most multi-storey composite steel floor fire engineered buildings in New Zealand
- This workshop presents the next revision to the third edition (ie the fourth edition)




Input screen 2006 edition


Output screen 2006 edition

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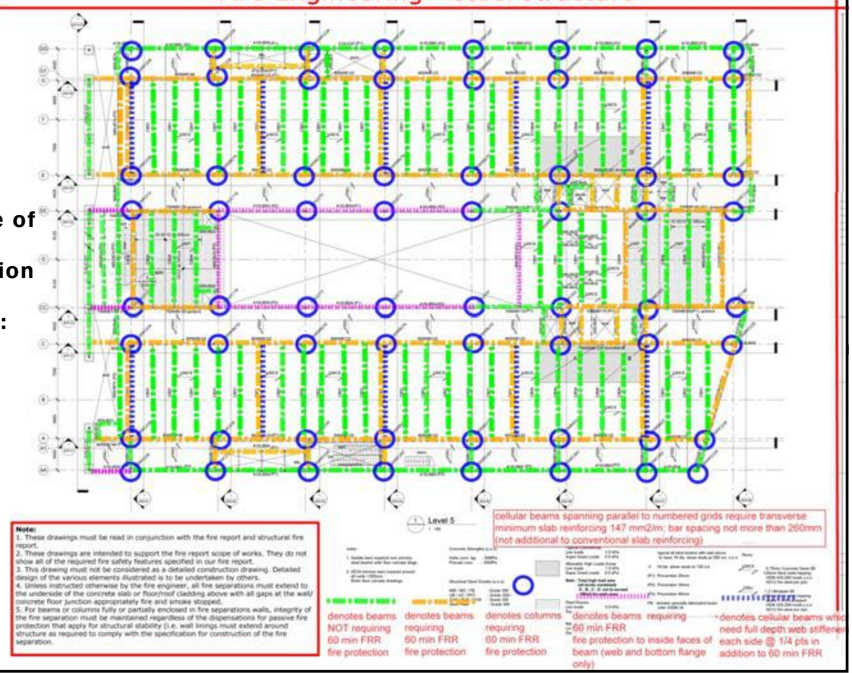
Fire Engineering - steel structure

Example of SPM application to office building: 2007



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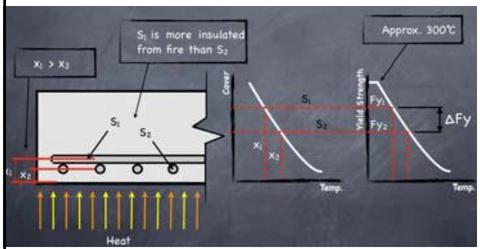
Step 14: Incorporating Orthotropic Reinforcement Conditions into Tensile Membrane Model 2008/2009

- 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



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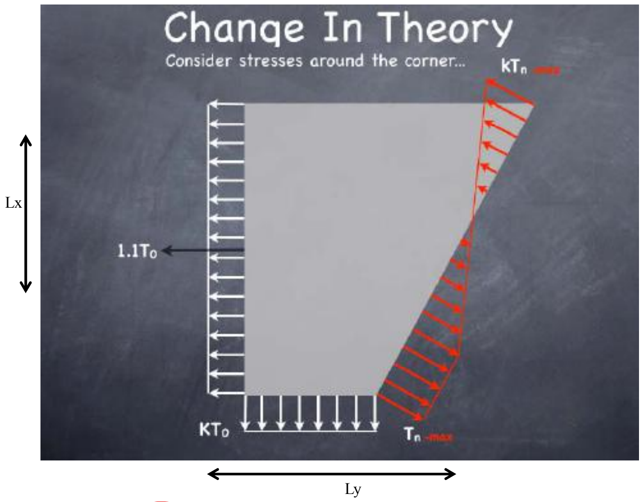
Step 15: Improving the Accuracy of the Tensile Membrane Model 2009

- 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



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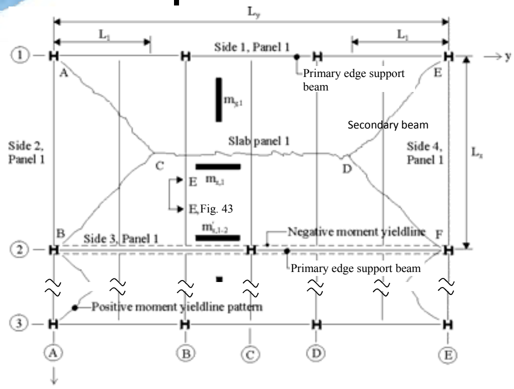
Step 16: Consideration of "double dipping" in regard to tension action in slab panel

- 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

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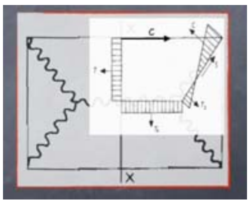
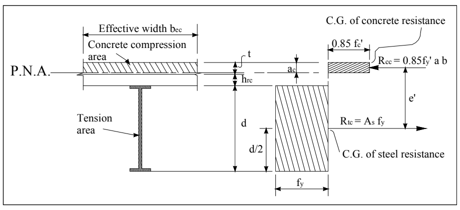
Step 17: Including Limitation Based on Compression Failure of Concrete Compression Ring 2010

- 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
- More on this in the application slides





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Step 18: Critical Review of Design Temperatures of Unprotected Secondary Beams within Slab Panel and SPM Deflection Limits 2011

4th year student project in 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

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Step 18: Critical Review of Design Temperatures of Unprotected Secondary Beams within Slab Panel and SPM Deflection Limits 2011

Fire test	$\phi_{fire}w_u$	w^*_{test}	$w^*_{test}/\phi_{fire}w_u$	Δ_{limit}	Δ_{test}	$\Delta_{test}/\Delta_{limit}$	t_{eq}	Notes on t_{eq}
	kPa	kPa		mm	mm		mins	
Cardington Furniture Test	7.09	4.94	0.7	726	642	0.88	54	Calculated from $t_{eq} = e_f k_b w_f$
Cardington Corner Test	6.47	4.94	0.76	754	388	0.51	62	Calculated from $t_{eq} = e_f k_b w_f$
Cardington 2003 Test	5.25	7.15	1.36	777	919	1.18	57	Calculated from $t_{eq} = e_f k_b w_f$
Mokrsko Test	7	6.6	0.94	864	892	1.03	65	Calculated from $t_{eq} = e_f k_b w_f$
FRACOF Test	19.55	6.89	0.35	750	460	0.61	120	Duration heating curve in furnace
COSSFIRE Test Option 1 (Note 1)	8.91	6.41	0.72	668	465	0.7	120	Duration heating curve in furnace
COSSFIRE Test Option 2 (Note 1)	4.19	6.41	1.53	668	465	0.7	120	Duration heating curve in furnace
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.

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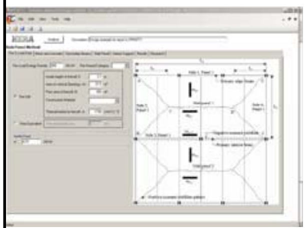
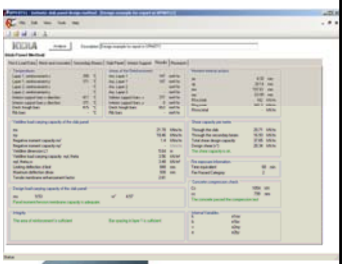
Step 19: Rewriting of SPM Software 2011 to 2012

- 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 with ongoing QA 2013/2014
- Incorporates all stages of development
- Demonstration to follow





Step 20: Comparison of SPM with Other Desktop Based Computer Programs for Composite Floor System Design

- Summer research project 2012/2013 (Daniels 2013)
- Comparison SPM, MACS+, TSLAB
- Conclusions:
  - SPM is the most comprehensive and technically accurate
  - SPM is the only one including detailing requirements
  - SPM and TSLAB bases design adequacy on structural fire severity (t_e)
  - MACS+ bases design adequacy on either structural fire severity or parametric time temperature fire exposure

Step 21: Strength and Stiffness of Slab Panel Edge Support Beams

- Part 4 Student Project 2013 (Su, Zhang, 2013)
- Also MEFE project
- Findings:
  - Slab panel support beams must have sufficient strength and stiffness to avoid a plastic collapse mechanism
  - Maximum support beam deflection < span/75 for effective slab panel support
  - Some changes to support beam loading
  - See application slides

Step 22: Modification to Slab Panel Deflection Limits

The deflection limits given in HERA Report R4-131 equations A23.3, A23.4 and A23.6 are modified to the following:

$$\Delta_{limit} = \left[min(\Delta_1;\Delta_2) - 0.5\left({}^{L_{xb}}/_{100} + {}^{L_{yb}}/_{100}\right)\right] C_{ISO} \le {}^{L_x}/_{15}$$

Revised A23.3

$$C_{ISO} = 0.0074 t_{eq} + 0.63 \ge 0.9$$

Revised A23.4

$$\Delta_{max} = min(\Delta_1;\Delta_2)\, C_{ISO} + \Delta_{spsb}$$

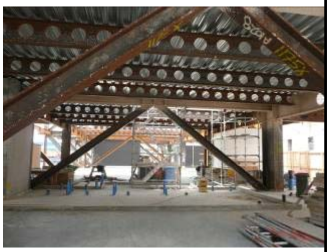
Revised A23.6

Step 22: Reasons for deflection limit modifications

- Eqn A23.3 – slab panel support beam deflection reduces tensile membrane enhancement; based on average deflection along parabolic deflected shape
- Eqn A23.3 – span/15 is slightly less than limit that has been tested to without failure
- Eqn A 23.4 – see details in (Wu et al, 2012)
- Eqn A23.6 – gives total deflection that floor may reach for determining required clearance underneath for fire separating walls running under middle of slab panel

Potential Future SPM Related Research

Contribution of Long Span Beams with Continuous Web Openings to Slab Panel Resistance



- These are becoming more common
- Status:
  - web contribution currently ignored
  - bottom flange laterally buckles
  - is this accurate?
- Need student and funding

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Slab Panel Performance with Steel Fibre Reinforcement

- General determination following on from 2011 research
- 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?

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Determining the Adequacy of Slab Panel Detailing Provisions

- 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
- Planned second VUT test imminent that will test some of these provisions further especially the strength and stability of support beam requirements

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Modifications to the Application of the SPM:2006 Edition and Application to C/VM2

By G Charles Clifton,
University of Auckland
and
Anthony Abu
University of Canterbury

1

Scope of Presentation

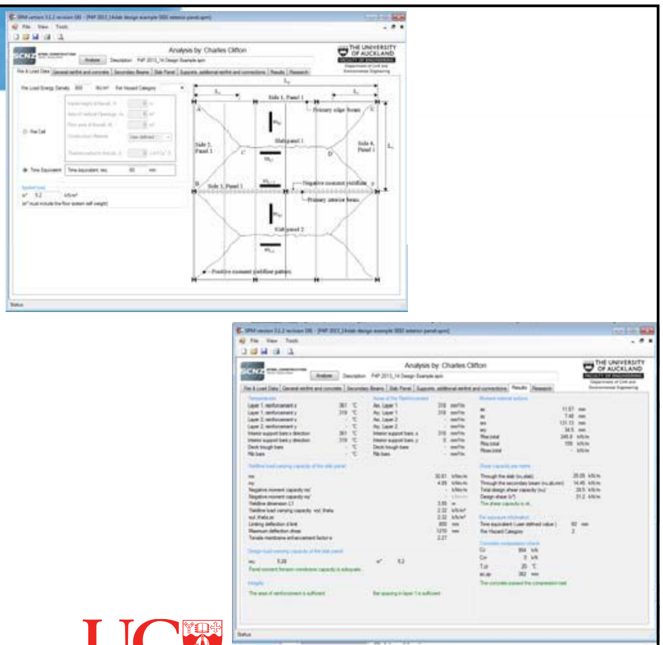
These slides cover:

- Changes to 2006 edition regarding implementation
- How to implement new software: this is covered by worked examples in second half of presentation
- Modification of HERA Report R4-131: 2006

2

SPM software

- Major Rewrite
- Much more user-friendly
- Multiple input screens
- Diagrams to guide determination of input
- Expanded printed output



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Detailing of Slab Panel Reinforcement: 1 of 2

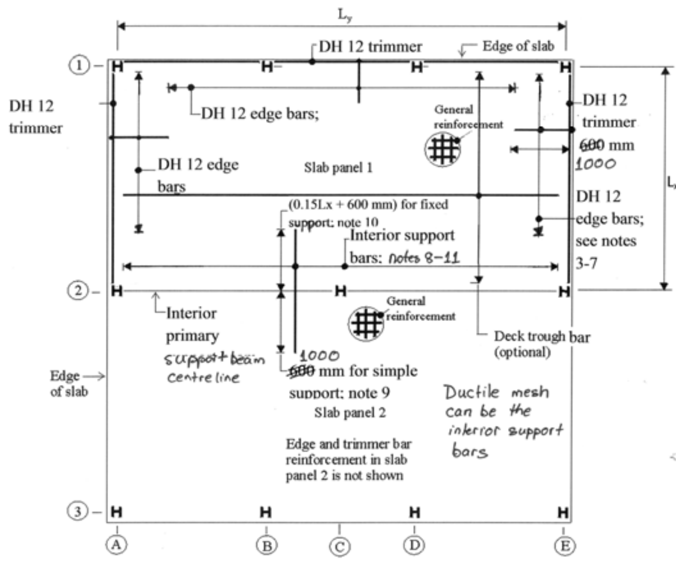

Figure 8 modified from HERA Report R4-131

Detailing of Slab Panel Reinforcement: 2 of 2

Reasons for Changes:

- Trimmer bar length increased to suppress shear fracture near supports observed in large scale Australian (VU) fire test
- Layout of trimmer bars in corners modified so only one layer specified; otherwise too much congestion of reinforcement
- Ductile mesh is now standard practice and can be used as interior support bars

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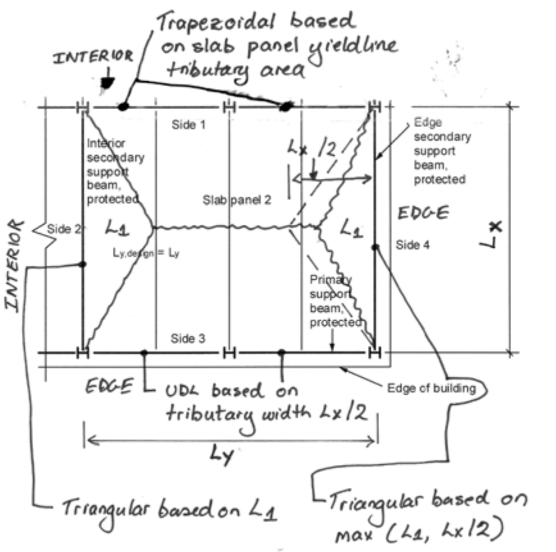
Increased Loading on Slab Panel Support Beams Along Edges of Building: 1 of 2

Slab panel support beams along the edges of a building require enhanced loading as shown. Applies to beams at the physical edge of a slab.



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Increased Loading on Slab Panel Support Beams Along Edges of Building: 2 of 2

Reasons for Changes:

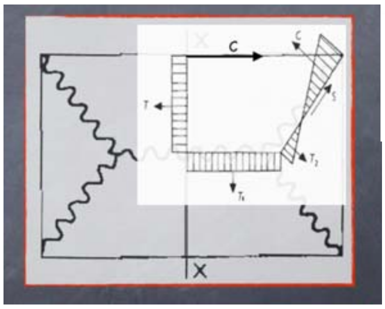
- Study on slab panel stability 2013 (Su, Zhang 2013) showed edge beams designed for loads based on yield line tributary area start to form plastic collapse mechanism before the specified FRR (time equivalent) period is achieved.
- Only an issue for edge beams; slab panel interior support beams can be designed for loading from slab panel yield line tributary area

Restraint from End Connections to Slab Panel Support Beams

- Deflection of support beams < span/75
- Simple connections cannot develop moment resistance to the beam in fire
- Semi-rigid and rigid connections can develop moment capacity based on same load paths as for ambient temperature design

Suppression of Concrete Slab Edge compression failure: 1 of 3

- Tensile membrane action can generate concrete compression failure at middle of long edge
- Concrete slab in this region may also be resisting composite action from slab panel support beam
- Need to account for both effects to avoid overstressing concrete



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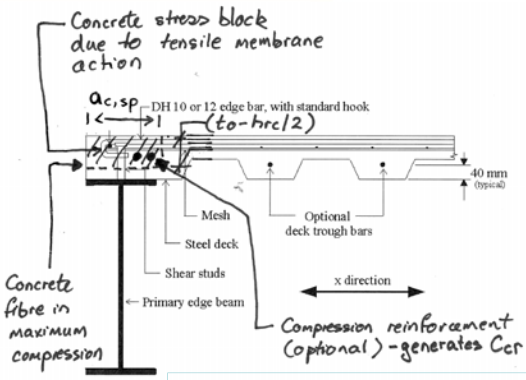
Suppression of Concrete Slab Edge compression failure: 2 of 3



Output from SPM

Concrete compression check

Cc	954	kN
Ccr	314	kN
T,cr	319	°C
ac,sp	256	mm

The concrete passed the compression test

C_c = compression force from tensile membrane action

C_{cr} = compression carried by compression reinforcement

$a_{c,sp}$ = depth of concrete compression stress block generated by tensile membrane action < that associated with compression failure

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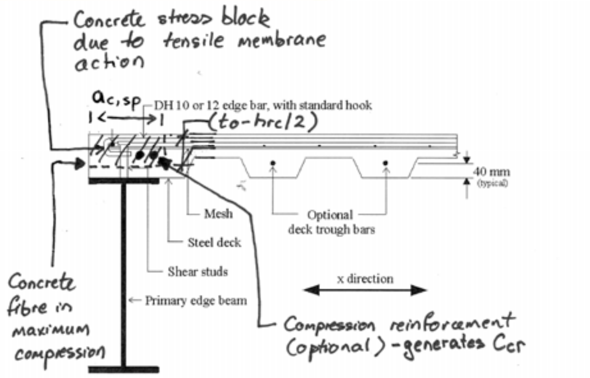
Suppression of Concrete Slab Edge compression failure: 3 of 3



If the support beam is resisting the loads by composite action then $a_{c,sp}$ must be deducted from the effective width of the concrete slab required for composite action as the compression from each is generated by different mechanisms and is additive. This is where the compression reinforcement can be placed to resist the tensile membrane induced compression

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References for SPM Modifications to Application

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Application to C/VM2

- SPM is a design procedure based on resistance to fully developed fire
- Three options for fully developed fire given by C/VM2. These are:
  1. Use a time equivalent formula and ensure FRR ≥ 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
- SPM is used with the first option; or with a FRR from the C/AS set of Approved Documents

Modifications Proposed to C/VM2: 1 of 4

- A new joint Australasian Composite Standard, AS/NZS 2327, is under development.
- Draft for public comment due for completion end 2014
- New section 6 on fire proposes two important modifications to C/VM2. These are as detailed on the next 3 slides

Modifications Proposed to C/VM2: 2 of 4

First modification is to the time equivalence equation:

$$t_e = e_{f,\mathrm{mod}} k_b k_m w_f$$

No 20 minute minimum value for steel or composite steel/concrete members

Reasons for first modification:

1. The equations have been developed for protected steel
2. The km factor accounts for the faster heating rate of unprotected steel
3. There is no modification in the Eurocode application of t_e
4. C/VM2 applies it to other materials for which a modification may be appropriate

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Modifications Proposed to C/VM2: 3 of 4

Modification to the fire load modification factor, $F_{m,}$ used to calculate $e_{f,mod}$ used in the t_e equation

Remove the distinction on ductility (all steel structures designed and detailed to our earthquake requirements will have dependable deformation capacity in fire)

Replace with :

- F_m = 1.0 for unsprinklered buildings
- F_m = 0.5 for sprinklered buildings where the fires are localised and the fire load is not more than 400 MJ/m^2 floor area (examples are car park fires, hotels and motels)
- F_m = 0.5 for other sprinklered buildings with an escape height of < = 10m
- F_m = 0.75 for other sprinklered buildings with an escape height > 10m but < = 25m.
- F_m = 1.0 for other sprinklered buildings with an escape height > 25m

Modifications Proposed to C/VM2: 4 of 4

Reasons for proposed F_m modifications:

1. This should be a modification only to the loadings side of the $S^* \leq \phi R_u$ equation
2. With sprinklers, the fire load can be taken as the "arbitrary point in time" (APT) fire load to be used if sprinklers don't suppress the developing fire
3. The APT fire load is typically 0.6 to 0.75 x the 80% fire load
4. For buildings with isolated fires, benefit of the localised nature of the fire is also recognised in $F_m = 0.5$
5. For low-rise buildings, some benefit of Fire Service intervention is included in reduction to $F_m = 0.5$
6. Where fire service can reach floors from the outside, upper value of fire load from 3 is proposed, ie $F_m = 0.75$
7. Above that height, no reduction in fire load applies, ie. $F_m = 1.0$

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