FIRE DESIGN AND SERVICEABILITY ISSUES – METAL DECK SLABS SUBJECT TO CONCENTRATED LOADING

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Abstract
Composite metal deck slabs consist of a profiled steel decking and an insitu concrete topping. The decking serves the dual function of permanent formwork to the wet concrete and when the concrete has gained sufficient strength, as external reinforcement to the slab to resist in-service loads.

In some applications such as bulk retail projects the metal deck floors must be capable of sustaining heavy concentrated loads from racking systems and goods handling equipment such as forklift trucks. Amongst other things, two important design issues need to be considered in these situations. Firstly the floor system must remain stable during emergency fire conditions and secondly the slab deflections must stay within acceptable limits.

In this paper the focus is on procedures suitable for design office practice to address both of these issues with particular reference to slabs subject to concentrated loading. Composite slabs can be designed as either simply supported or continuous members. The design guidance provided is written for simply supported span arrangements. The use of these procedures is illustrated by way of worked example.

Introduction
The material in this paper draws on previous SCNZ and HERA reports (Fussell et al 2006, Barber and Clifton 1994 and Clifton 2006) which cover the ultimate limit state design of metal slabs subject to concentrated loading and fire engineering design. The intention is to cover issues not addressed by any of these previous documents, namely fire design and deflection limits for metal deck slabs subject to concentrated loading. The emphasis is on directing structural engineers to appropriate sources of design guidance and where required, how to modify this design guidance to address fire design and serviceability issues. To make this document easy to read alongside these earlier publications, the original notation where possible is used.

The SCNZ Design Guide for Concentrated Loads on Metal Deck Slabs (Fussell et al 2006) is based on the British Standard, Code of Practice for Design of Composite Slabs with Profiled Sheeting (BS 5940 Part 4 1994) which has a slightly different limit state procedure to that used in New Zealand. The principal difference is that British Codes (BS 5940.4 1994, BS8110.1 1997) use partial material factors of safety γ_m applied to each component of the section (concrete, reinforcing steel) as opposed to the New Zealand practice of applying a global strength reduction factor ϕ to the nominal capacities. The limit state approach presented in the SCNZ report (Fussell et al 2006) has been used in this paper. This has necessitated some minor modifications to the fire design methodology outlined in the HERA reports (Barber and Clifton 1994, Clifton 2006). These are discussed further in the Fire Design section.

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Fire Design

Introduction
At ambient temperature the metal decking acts compositely with the concrete slab to resist positive bending moments. When the metal deck sheeting is heated, as occurs when the slab is exposed to severe fire, its mechanical properties such as the yield strength and modulus of elasticity decrease. Once a critical temperature is reached, which is dependent on the load level and span arrangement, the decking is no longer able to provide the necessary tensile resistance to maintain the load carrying capacity of the slab (Barber and Clifton 1994).

Metal deck slabs with an average thickness greater than 60mm typically have an inherent 30 minute fire rating (Barber and Clifton 1994). If additional fire rating beyond this time limit is necessary, other measures such as additional reinforcement, fire rated suspended ceilings or the application of insulating coatings is required. In this paper only the provision of increased fire rating by means of reinforcing bars is considered. In this instance the slab design proceeds as for conventional reinforced concrete with due account taken of the reduced reinforcing bar yield strength (Figure 1).

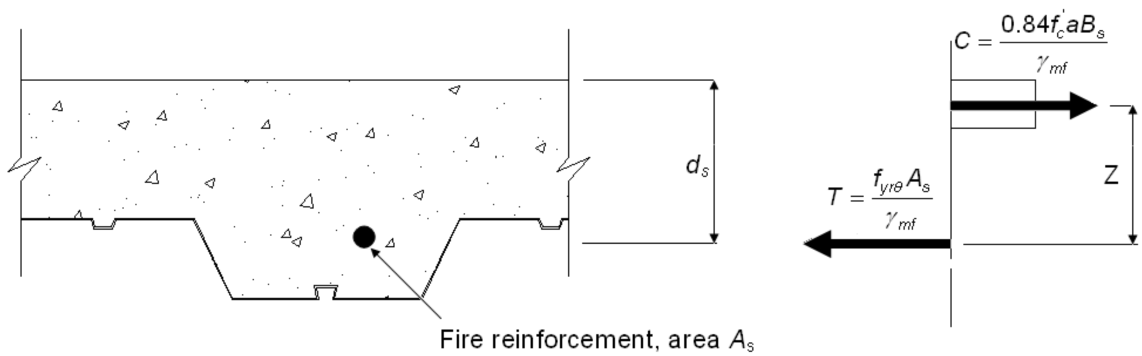

Figure 1: Internal actions - positive moment of resistance –emergency fire conditions

Additional reinforcing bars provided in the troughs of the decking will also help the ultimate limit state behaviour of the slab at ambient temperature. This is particularly so for slabs subject to high concentrated loads located close to supports where the bond between the concrete and the decking may not develop rapidly enough to mobilise the required tensile capacity of the decking. Reinforcing bars located in the ribs, provided they are adequately anchored, can be accounted for in strength design calculations.

In addition to possessing adequate strength and stiffness to maintain structural stability at elevated temperatures, the slab must also possess adequate integrity to stop flames and gases passing through the slab and insulation to limit the temperature on the unexposed side of the slab. Slabs that meet the insulation requirements, which are a function of effective concrete thickness, will also meet the integrity requirements.

For fire design the following strength reduction factors ϕ are used:

ϕ_{fire} = 1.0 (flexure) (Barber and Clifton 1994, Standard New Zealand 2006)
ϕ_{fire} = 0. 9 (shear) (Clifton 2006)

The rationale for the lower strength reduction factor associated with a shear failure mode is that this is a less desirable failure mode than flexure, and this is reflected in a lower ambient temperature strength reduction factor of 0.75 (shear) compared with 0.85 (flexure). To maintain the same relativity for fire design, ϕ_{fire} associated with shear failure has been determined as (Clifton 2006):

$$\phi_{fire,shear} = \frac{\phi_{shear}}{\phi_{flexure}} . \phi_{fire,flexure} = \frac{0.75}{0.85} x1.0 = 0.9 \qquad (1)$$

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If this philosophy is to applied to the design methodology in the Design Guide for Concentrated Loads on Metal Deck Slabs (Fussell et al 2006), some modifications are required to the limit state factors. For ambient temperature design the following parameters apply:

Table 1: Modified Design Equations and Material factors of Safety Relevant to Ambient Temperature and Fire Design

Design Action	Design Equations (BS 5950.4 1994 and BS 8110.1 1997)	γ_m Ambient temperature	γ_{mf} Fire conditions
Flexure	Reinforcing steel design yield stress $\frac{f_y}{\gamma_m}$	1.15	1.0
	Concrete compression stress block $\frac{0.84f_c^{'}}{\gamma_m}$	1.5	1.3
Shear	Concrete design shear stress: $v_c = \frac{0.79}{\gamma_m}\left(\frac{f_c^{'}A_s}{200d_s}\right)^{\frac{1}{3}}\left(\frac{400}{d_s}\right)^{\frac{1}{4}}$ (2) subject to: $\frac{A_s}{d_s} < 30$ and $\left(\frac{400}{d_s}\right)^{\frac{1}{4}} \geq 0.67$ and $f_c^{'} < 32$ MPa If $f_c^{'} > 32$ MPa, compute v_c using $f_c^{'} = 32$	1.25	1.1

The equations in table 1 have been slightly modified from the original documents. Concrete cube strengths fcu have replaced with equivalent cylinder strengths using the relationship, $f_c^{'} = 1.25f_{cu}$. The concrete compression stress block formulae has been derived by multiplying the equivalent stress block ($0.67f_{cu}$) which is used in the British Concrete Code (British Standards Institute 1997) by the cylinder strength conversion factor. The term A_s is the area of additional slab reinforcement for fire design or the decking area for ambient temperature design. The term d_s represents the slab effective depth which is the distance from the fire rebar or decking centroid to the concrete top surface (positive bending).

In keeping with New Zealand fire design practice, the material factor of safety for the reinforcing bars, γ_{mf}, has been set to unity. To maintain the same relationships between the various material factors of safety at elevated temperature, the material factors of safety for the concrete stress block and concrete shear have been derived by dividing their ambient temperature material factor of safety by a factor of 1.15 (rebar γ_m), see Table 1.

Slab Failure Model

The load leading to slab failure at elevated temperatures can be computed using plastic theory (Barber and Clifton 1994). The magnitude of this failure load will be dependent on the number of plastic hinges that are assumed to form and the plastic moment capacity of each of these hinges. In this paper a simply supported slab arrangement is assumed. For this arrangement, the limiting slab failure load occurs when the maximum positive moment in the span reaches the slab plastic moment capacity, see figure 2.

Based on this failure mechanism, the limiting load condition including a single midspan concentrated load is:

$$w^* \frac{L_s^2}{8} + \frac{P_{eb}^* L_s}{4} \leq M_n \qquad (3)$$

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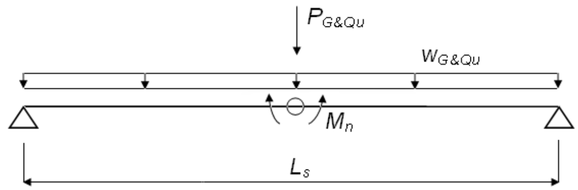

Figure 2: Simply supported elevated temperature failure mechanism

Where w* and P^*_{eb} are the factored uniformly distributed and concentrated loads per metre width of slab respectively. To determine P^*_{eb} refer to equation 15. The load combination factors appropriate for emergency fire conditions from the Loadings Standard AS/NZS1170.0 (Standards Australia, Standards New Zealand 2002) are:

[G, ψ_l Q]

G is the permanent load, Q is the full imposed load and ψ_l is the long term load factor.

The positive plastic moment capacity associated with stress distribution shown in figure 2 is:

$$M_n = \frac{A_s f_{yr\theta}}{\gamma_{mf}} . \left(z - \frac{a}{2} \right) \quad (4)$$

$$a = \frac{A_s f_{yr\theta}}{0.84(f'_{c20} / \gamma_{mf})B_s} \quad (5)$$

; the appropriate γ_{mf} values are found in Table 1.

A_s is the area of trough reinforcing bars per metre width of slab, B_s is the width of slab, a is the depth of concrete compression stress block and $f_{yr\theta}$ is the reduced yield stress accounting for the elevated temperature of the reinforcing bars. The procedure for computing $f_{yr\theta}$ is presented in appendix A of "The Design of Composite Steel Floor Systems for Severe Fire" (Clifton 2006). The derivation of the equivalent stress block, $0.84f'_c$ is discussed in the preceding section.

Implicit in these formulae are the following simplifying assumptions:

- The metal deck sheeting does not contribute to flexural capacity of the slab at elevated temperatures
- The strength of the concrete compression zone, which is remote from the fire, will not be influenced by the fire allowing ambient temperature properties to be used (f'_{c20}). This will be true when the fire resistance rating is the same for stability, integrity and insulation. If the insulation resistance rating is less than the stability requirement, this assumption should be checked using the procedure in HERA report R4-82 (Barber and Clifton 1994).

Even though metal deck slabs are designed as one way spanning, concentrated loads induce local bending in the transverse direction as an effective width of slab resisting such loading is mobilized. The magnitude of this bending can be computed using the design equation found in the publication, Design Guide for Concentrated Loads on Metal Deck Slabs (Fussell et al 2006). The elevated temperature transverse flexural capacity of the slab can be determined using the principles just discussed. This includes the use of the reduced design yield stress $f_{yr\theta}$ and the appropriate material factors of safety γ_{mf} found in Table 1.

Slab Shear Capacity

The shear capacities of composite slabs are computed using the same empirically derived formulas as for conventionally reinforced concrete (Oehler and Bradford 1995). During emergency fire conditions the contribution of the decking is neglected. The concrete design shear capacity of simply supported slabs should therefore be computed using only the area of the additional reinforcing steel located in the profile troughs.

The necessary design criteria to avoid slab shear failure for a single concentrated load, (Figure 2) is:

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$$V^* = w^*\left(\frac{L}{2} - d_s\right) + \frac{P_{er}^*}{2} < V_v \qquad (6)$$

The design shear capacity V_v, for ribbed and flat deck profile slabs can be found using the equations in the SCNZ design guide (Fussell et al 2006). The required design shear value v_c is computed using equation 2 in Table 1, and the appropriate material factor of safety for fire conditions. To determine P_{er}^* refer to equation 15.

Punching Shear Capacity
The punching shear capacity of a metal deck slab (V_p) at elevated temperature is found using the following relationship.

$$V_p = b_p\left(D_s - D_p - e_\theta\right)v_c \qquad (7)$$

Where:

$$b_p = 2\left[\left(b_0 + 2\left(D_s - D_p - e_{c\theta}\right)\right) + \left(d_s + b_1\right)\right] \qquad (8)$$

e_θ = 14 mm for fire hazard category 1 (Clifton 2006)
= 17mm for fire hazard category 2
= 19mm for fire hazard category 3

$$v_c = \frac{0.79}{\gamma_{mf}}\left(\frac{f_c' A_{sp}}{200\left(D_s - D_p - e_\theta\right)}\right)^{\frac{1}{3}}\left(\frac{400}{D_s - D_p - e_\theta}\right)^{\frac{1}{4}} \qquad (9)$$

subject to:

$$\frac{A_{sp}}{\left(D_s - D_p - e_{c\theta}\right)} < 30 \text{ and } \left(\frac{400}{\left(D_s - D_p - e_{c\theta}\right)}\right)^{\frac{1}{4}} \geq 0.67$$

and $f_c' < 32$ MPa

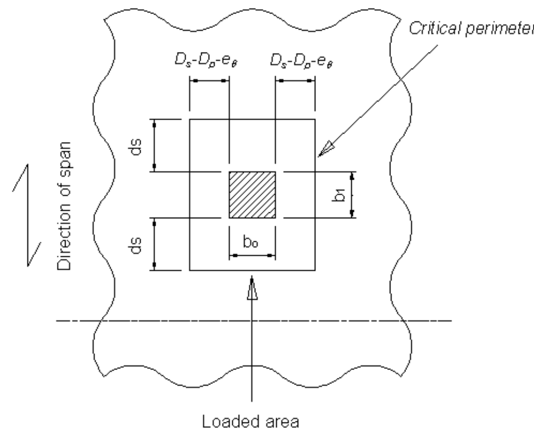

Figure 3: Critical perimeter punching shear

The terms D_s and D_p represent the overall slab depth and the depth of the metal deck profile. The concrete design shear stress v_c is computed using the γ_{mf} value found in Table 1 and A_{sp}, the weighted average of topping slab rebar in both principal directions which cross the potential punching shear failure surface. The term e_θ accounts for the loss of strength of the surface concrete layer (underside of slab), which following exposure to elevated temperatures is considered ineffective in providing resistance to punching loads. This term is a function of the fire hazard category of the structure. Fire hazard categories are defined in the Approved Document for New Zealand Building Code Fire Safety Clauses (Building Industry Authority 2001).

Minimum Slab Thickness For Insulation
The insulation fire resistance criterion is satisfied if the average temperature increase on the face remote from the fire does not exceed 140°C. To achieve this requirement sufficient depth of concrete is necessary. The minimum slab effective depths to meet the insulation requirements for various fire resistance ratings are found in the Concrete Structures Standard NZS3101 (Standards New Zealand 2006). For ribbed profiles the effective depth h_e is a function of the rib geometry and the overall concrete thickness. In most instances, except for deeper ribbed profiles, h_e is the average concrete thickness.

Deflection Checks

Introduction
Accurately predicting the deflection of concrete structures is difficult due to the non linear behaviour of concrete even under service loads (Gilbert 2001). This non linear behaviour is due to the effects such as cracking, tension stiffening (the stiffening effect of intact concrete between cracks), creep and shrinkage. Approaches for accounting for these effects are discussed in the following sections.

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Concrete Cracking/ Tension Stiffening

The method adopted in the British composite slab standard (British Standards Institution 1994) to account for this effect is take a single effective moment of inertia which is the average of the cracked and uncracked composite slab properties using a transformed steel areas approach. Decking manufacturers design manuals typically present uncracked (I_g), cracked (I_{cr}) and average composite slab properties (I_{ave}) for various slab thicknesses and modular ratios.

$$I_{ave} = \frac{1}{2}\left(I_{cr} + I_g\right) \qquad (10)$$

In this paper properties calculated using a short term concrete modulus E_c have been designated I_{aves},I_{crs},I_{gs} while those computed a long term concrete modulus E_{cr} have been designated I_{avel}, $I_{crl.}$ and I_{gl}.

Creep

The simplest method for assessing long term creep effects is the effective modulus method. With this approach the concrete short term modulus E_c is modified to account for the additional strains that accompany sustained loading (Bryant et al 1984). This relationship is of the form:

$$E_{cr} = \frac{E_c}{1 + C_t} \qquad (11)$$

To account for creep, deflections resulting from permanent loading are computed using long term composite moment of inertia I_{avel}. In keeping with the recommendation of the Steel Structures Standard Commentary NZS3404 Part 2 (Standards New Zealand 1997), a ratio of elastic to long term concrete modulus of 2.5 is suggested. This equates to a creep factor C_t of 1.5.

Shrinkage

Tensile stresses are induced in concrete sections if they are not free to shrink without restraint. This restraint can be from external sources such as support members and adjacent structures, or internally by bonded reinforcement and in the case of composite slabs, the metal decking. If the reinforcement is not placed symmetrically in the section a shrinkage induced curvature develops with time. As a result, downward deflection can even occur in an otherwise unloaded concrete member (Gilbert 2001).

The magnitude of the shrinkage induced deflection can be computed using the following formula from the commentary to the Steel Structures Standard NZS 3404 Part 2 (Standards New Zealand 1997).

$$\Delta_{sh} = \frac{\varepsilon_{sh} E_c A_c L_s^2 e_c}{8 E_s I_{gs}} \qquad (12)$$

The terms are as follows: ε_{sh} is the concrete shrinkage strain, typically taken as 300 microstrain for composite metal deck slabs; E_c and E_s are the short term concrete and steel elastic modulus respectively; A_c is the uncracked area of concrete slab; L_s represents the span of slab; e_c is defined as the distance from the line of the shrinkage force action to the neutral axis of the composite slab. The neutral axis depth of the uncracked composite slab is computed using the steel to concrete modular ratio E_s/E_c.

Equation 12 is based on a simply supported member subject to equal shrinkage induced end moments of:

$$M_{sh} = \varepsilon_{sh} E_c A_c e_c \qquad (13)$$

Deflection Limits

The appropriate long and short term load duration factors ψ_l, ψ_s found in the Loadings Standard AS/NZS 1170.0 (Standards Australia/ Standards New Zealand 2002), are used to compute serviceability limit state live loads for deflection calculations.

When concentrated loading is present, the serviceability limit state load per metre width of slab, P^*_{ebs}, should be computed using the effective width b_{eb}, (Fussell et al 2006) and is determined as follows.

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$$P^*_{ebs} = \frac{P^*}{b_{eb}} \tag{14}$$

The b_{eb} value used, which is defined as the width of slab effective in resisting concentrated loading, should be appropriate for the span arrangement assumed; either simply supported or continuous. For simplicity sake, the slab can initially be checked as simply supported. If the deflections are excessive, and the designer does not want to increase the slab depth, or change the slab span L_s , the beneficial effect of slab continuity can be considered.

If continuity is allowed for, the calculations become more onerous. The slab will now likely have different moments of inertia due to differing tension reinforcing steel contents in the positive and negative moment regions which needs to be accounted for in any assessment of continuous slab deflections. This can be achieved using the recommendations of concrete codes such as NZS3101 (Standards New Zealand 2006) which specifies how to model continuous elements of structure with variable stiffness. Creep effects can be also be accounted for using the long term modulus E_{cr} previously discussed in this paper . Calculation of the shrinkage induced deflection is also a more complex matter with slab continuity. In addition to the uniform shrinkage moment M_{sh} which develops as a result of restraint to shrinkage, secondary bending moments are also induced to maintain displacement compatibility of the slab at the supports. These actions reduce the midspan shrinkage deflection. Therefore, it is suggested as a simplified approach to compute shrinkage deflections on the basis of simple support conditions even if the remaining components of deflection are computed assuming slab continuity.

The following recommended deflection limits are taken from the New Zealand Loadings Standards AS/NZS 1170.0 (Standards Australia and Standards New Zealand 2002) and NZS 4203 (Standards New Zealand 1992).

Table 2: Deflection Limits for Composite Slabs

Control Phonomenon	Propping Arrangement	Deflection Limit	Deflection components from:	Action considered
Visual	Propped	L/300	$G_c + G_{SDL} + G_p + \psi_l Q + \Delta_{cr} + \Delta_{sh}$	Ripple
	Unpropped		$G_{SDL} + G_p + \psi_l Q + \Delta_{cr} + \Delta_{sh}$	
Protection of non structural elements	-	L/300	$G_{SDL} + +\psi_s Q + \Delta_{cr} + \Delta_{sh}$	Framed gypsum wall cracking
	-	L/500		Masonry wall cracking
Functionality	Propped	L/250	$G_{c+} G_p + G_{SDL} + \psi_s Q + \Delta_{cr} + \Delta_{sh}$	Floor sagging
	Unpropped		$G_p + G_{SDL} + \psi_s Q + \Delta_{cr} + \Delta_{sh}$	

Notation

G_c Permanent load of decking and wet concrete including any ponding

G_{SDL} Super imposed permanent loading excluding partitions

G_p Partition loading

$\psi_s Q$ Short term portion of imposed load (AS/NZS 1170.0 2002)

$\psi_l Q$ Long term portion of imposed load (AS/NZS 1170.0 2002)

Δ_{cr} Concrete creep deflection under long term loading

For unpropped construction, creep inducing load consists of long term superimposed load on the composite section $(G_p + G_{SDL} + \psi_l Q)$

For propped construction, the creep inducing load should also include slab self weight $(G_c + G_{SDL} + G_p + +\psi_l Q)$

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Notes

- The concrete surface is assumed to be screeded level
- The creep term Δ_{cr} is only calculated on the long term component of loading
- The modular ratio used to compute composite section properties should be appropriate for the load duration ie for short term loads use E_s/E_c, for long term loads use E_s/E_{cr}
- The designer should satisfy themselves that these limits are applicable for their particular project.

Concentrated Loading

The approach taken in the British composite slab standard (British Standards Institution 1994) to modeling the load effects from concentrated loading is to determine a width of slab which is considered effective in resisting the design actions (shear and flexure) resulting from such loading. These effective widths differ for shear and flexure and have been designated in this paper as b_{er} and b_{eb}. Formulae for computing these variables are found in the SCNZ design guide (Fussell et al 2006). The concentrated load may then be divided by the effective width to give the concentrated load applied to a unit width of slab.

$$P_{eb} = \frac{P}{b_{eb}} \text{ or } P_{er} = \frac{P}{b_{er}} \tag{15}$$

Worked Example

To illustrate the use of the design procedures for fire design and deflection checks, a 130mm HiBond (Dimond 2004) slab will be checked for a combination of uniformly distributed and concentrated loading (loaded area 115mmx115mm). A fire resistance rating of 90 minutes is required (fire hazard category 2).

Floor data

Floor system:	130mm thick HiBond
Floor span:	2.6m (unpropped)
b_a	156 mm (average trough width)
b_r	307 mm (trough pitch)

Concrete Data

f_c'	30 MPa
E_c	25 GPa (short term modulus)
E_{cr}	10 GPa (long term modulus)
n_s	8.2 (short term modular ratio)
n_L	20.6 (long term modular ratio)

Load Data

Permanent

w_c	2.57 kPa (slab weight, 5% ponding)
w_{SDL}	0.5 kPa (finishes, ceiling, services)

Imposed ($\psi_s = 0.7$(UDL), $\psi_s = 1.0$(conc.), $\psi_l = 0.4$)

w_Q	1.5 kPa
w_p	0.3 kPa (partitions)
P_Q	15 kN

Section Properties (moments of inertia based on an equivalent transformed steel section)

Concrete only		Short term properties				Long term properties		
A_c mm^2	y_c mm	y_{ts} mm	I_{gs} x10^6 mm^4	I_{crs} x10^6 mm^4	I_{aves} x10^6 mm^4	I_{gl} x10^6 mm^4	I_{crl} x10^6 mm^4	I_{avel} x10^6 mm^4
102948	54.7	58.5	17.8	7.15	12.4	8.53	5.43	6.98

Note: y_c and y_{ts} are measured from the top of the slab (neutral axis depth). I_{gs} and I_{gl} are the uncracked transformed moments of inertia using short and long term modular ratios respectively. I_{crs} and I_{crl} are the cracked composite short and long term moments of inertia. I_{aves} and I_{avel} are the corresponding average section properties (moment of inertia).

Concentrated Loading per metre width of slab

Effective width resisting:		Fire design $G+\psi_l Q$		Serviceability	
b_{eb} (flexure) m	b_{er} (shear) m	P_{ebf} kN	P_{erf} kN	P_{ebs} kN	P_{ebl} kN
1.47	0.9	$\frac{15x0.4}{1.47} = 4.1$	$\frac{15x0.4}{0.9} = 6.7$	$\frac{15x1}{1.47} = 10.2$	$\frac{15x0.4}{1.47} = 4.1$

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The terms P_{ebf} (flexure)and P_{erf} (shear) are the unfactored concentrated loads supported by a one metre wide strip of slab. These are used to derive deriving flexural and shear design actions induced by such loading during a fire event. P_{ebs} and P_{ebl} are the unfactored concentrated loads applied to a one metre width of slab for the purpose of computing short and long term deflections.

Load combination summary

Fire $G+\psi_l Q$			Deflections			
w^* kN/m	P^*_{eb} kN	P^*_{er} kN	$w^*_{G_p+G_{SDL}+\psi_s Q}$ kN/m	$w^*_{G_p+G_{SDL}+\psi_l Q}$ kN/m	$P^*_{\psi_s Q}$ kN	$P^*_{\psi_l Q}$ kN
3.97	4.1	6.7	1.85	1.4	10.2	4.1

Fire Design

Check minimum slab thickness for insulation. Refer to the following figure (Figure 3) for notation definitions.

h_1 mm	h_2 mm	l_1 mm	l_2 mm	l_3 mm	$h_e = h_1 + \frac{h_2}{2}\left(\frac{l_1+l_2}{l_1+l_3}\right)$ mm	h_e min (NZS3101 2006) mm
75	55	182	130	126	102.9	95

Therefore a 130mm slab meets the minimum effective depth requirement to achieve a 90 minute insulation rating.

Design Actions

$$M^* = w^* \frac{L_s^2}{8} + \frac{P^*_{ebf} L_s}{4} = \frac{3.97x2.6^2}{8} + \frac{4.1x2.6}{4} = 6.02 \text{ kNm/m}$$

$$V^* = \frac{w^* L}{2} + \frac{P^*_{erf}}{2} = \frac{3.97x2.6}{2} + \frac{6.7}{2} = 8.51 \text{ kN/m}$$

Design flexural capacity at elevated temperature (10mm deformed bar per trough)

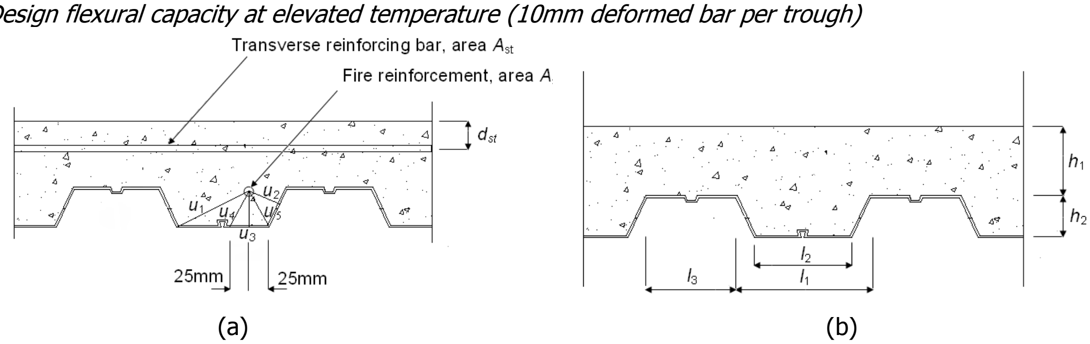

Figure 4: Notation definitions a) Heat transfer coefficient b) Effective thickness h_e

Determine elevated temperature rebar design yield stress.

Distance from rebar centroid to exposed slab faces					Heat transfer coefficent	Rebar temperature	Elevated temperature.design yield stress
u_1 mm	u_2 mm	u_3 mm	u_4 mm	u_5 mm	χ	$\theta\,^0C$	$f_{yr\theta}$ MPa
95	45	35	43	43	1.8 (*)	537 (*)	331 (*)

Figures noted (*) have been computed using equations found in reference (Clifton 2006).

Compute the slab design flexural strength using $f_{yr\theta}$

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Compression block depth	Rebar area	Lever arm	Flexural capacity	Design ratio
a mm	A_s mm^2/m	z (+) mm	M_n kNm/m	$\frac{M^*}{M_n}$
3.4	256	90.3	7.65	0.79

$$(+)\ z = \min\left[0.95d_s, \left(d_s - \frac{a}{2}\right)\right]$$

Design shear capacity at elevated temperature

Rebar area	Effective depth	Design shear stress	Slab shear capacity	Design ratio
A_s mm^2/m	d_s mm	v_c MPa	$v_v = \frac{b_a d_s v_c}{b_r}$ kN/m	$\frac{V^*}{V_v}$
256	95	0.77	37.2	0.23

Design punching shear capacity ($e_\theta = 17$ mm for fire hazard category 2)

Rebar area (665 mesh)	Effective depth of slab above deck rib	Design shear stress	Length of punching shear perimeter	Slab punching shear capacity	Design ratio
A_{sp} mm^2	D_s-D_p-e_θ mm	v_c MPa	b_p mm	$V_p = b_p v_c (D_s - D_p - e_\theta)$ kN	$\frac{P^*}{V_p}$
150	58	0.85	1072	53	0.11

By the addition of a single 10mm deformed bar per rib, a 130mm HiBond slab spanning 2.6m and subject to a 15 kN concentrated loading meets the stability, integrity and insulation requirements for a 90 minute fire resistance rating. The flexural capacity in the transverse direction would also need checking using a similar approach. Refer to the Slab Failure Model section of this paper for a discussion of transverse slab bending and a description of how this check is undertaken.

Deflection

Creep

$$\Delta_{cr} = \left[\frac{5w_{G_{SDL}+G_p+\psi_l Q} L_s^4}{384E_s} + \frac{P_{\psi_l Q} L_s^3}{48E_s}\right]\left[\frac{1}{I_{avel}} - \frac{1}{I_{aves}}\right]$$

Shrinkage

$e_c = y_{ts} - y_c = 58.5 - 54.7 = 3.8$mm

Deflection Summary

Short term (mm)		Long term (mm)	Shrinkage (mm)	Visual Criteria (mm)		Functionality Criteria (mm)	
$G_{SDL} + G_p + \psi_s Q$	$G_{SDL} + G_p + \psi_l Q$	Δ_{cr}	Δ_{sh}	Computed deflection	Deflection limit	Computed deflection	Deflection limit
1.95	0.95	0.73	0.7	2.38	8.6	3.38	10.4

The deflections of the 130mm HiBond slab spanning under the combination of uniformly distributed loads and

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concentrated loading are all within acceptable limits.

Conclusions

Metal deck slabs with an average thickness greater than 60mm have an inherent 30 minute fire resistance. If additional fire resistance rating is required, one option which may be considered is to provide additional bars in the slab. A design methodology has been presented for achieving the required fire resistance rating by way of additional reinforcement.

In addition to strength considerations, good serviceability performance, such as limiting slab deflections to acceptable limits is required. Design guidance has also been prepared to address this issue. The procedure accounts for the various non linear effects that occur in concrete structures such as cracking, creep and shrinkage.

Both of these procedures are particularly focused on metal deck slabs subject to concentrated loads which may be found in bulk retail type applications. The fire design procedure has been prepared for a simply supported span arrangement. However, this may easily be extended to cover continuous span arrangements.

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