STRUCTURAL FIRE DESIGN FOR STEEL FRAMED CARPARKS
Structural Fire Design for Steel Framed Carparks

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1. FORWARD

This publication is based on a Holmes Fire technical report *Structural fire design for steel framed car parks Version C* written by Dr Linus Lim and Martin Feeney and issued on 21st October 2015 to Steel Construction New Zealand. Earlier versions of the Holmes Fire technical report were reviewed by a working group of the following:

- Dr Anthony Abu, University of Canterbury
- Michael Belsham, Ministry of Business Innovation and Employment
- Associate Professor Charles Clifton, University of Auckland
- Kevin Cowie, Steel Construction New Zealand
- Alistair Fussell, Steel Construction New Zealand
- Dr Stephen Hicks, Heavy Engineering Research Association
- Dave McGuigan, Ministry of Business Innovation and Employment
- Simon Weaver, Aurecon
- Paul Williams, BECA

The Holmes Fire technical report was further reviewed by

- Dr Philip Xie, Opus International Consultants

This publication incorporates changes to address peer review comments.

The report was prepared prior to the publication of AS/NZS 2327:2017 Composite structures – Composite steel-concrete construction in buildings. No attempt has been made to update the report with reference to AS/NZS 2327:2017. Users are encouraged to refer to AS/NZS 2327:2017 for structural capacity equations. It is anticipated that future versions of this report will make reference to AS/NZS 2327:2017.
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1. INTRODUCTION

1.1 Overview
This document proposes an alternative methodology for calculating the inherent fire resistance of multi-bay, multi-storey steel framed carparking buildings with composite metal deck floor systems.

The proposed alternative method described here determines the capacity of steel beams within a carpark under exposure to realistic fires. The basis of the proposed methodology is from extensive research undertaken in Europe and Australia. Similar methodologies have been applied and developed in Europe.

A review and critique of the application of the Time Equivalence method for carparking buildings is provided in this document.

1.2 Impetus for this methodology
The objective of this proposed methodology is to provide a robust method of reducing / eliminating fire protection to structural steel members in multi-storey steel framed carparks whilst satisfying the Performance Requirements of the New Zealand Building Code.

1.3 Limit of application
The scope of application of the proposed method is limited to:

- Multi-storey steel framed carpark structures supporting composite metal-deck floor systems.
- Carpark structures (open or closed) that are used for passenger vehicles cars, vans and utility vehicles. It is not intended to be used for car stackers, loading docks and parking structures for heavy goods vehicles.
- The scope of application for this method applies only for beams and bracing elements.
- The basis for this method is that fire damage to beams and braces are likely to be localised to a floor, or part of a floor, with less impact to other floors above. Steel braces can be unprotected if the building lateral stability can be shown not to be affected during fire exposure and after fire exposure. It is noted that the braces may be expected to perform their lateral load resisting function to resist earthquake forces, after a fire, for instance in Christchurch.
- This method is currently not proposed to be applied to columns and requires that columns achieve the required Fire Resistance Rating either by passive fire protection, or if unprotected, by its inherent fire resistance. Beam connections are to be protected.
- Research into the HRR produced from hybrid and electric vehicles and vehicles with biofuels is currently not well documented but when the HRR data is obtained, it can be applied to the calculations.
- The heat transfer analysis applies for hot rolled structural steel members. Cold formed structural elements such as Speedfloor joists cannot be applied with this heat transfer method yet. Therefore, the method is currently proposed for hotrolled steel framed structures only.
- The method of calculation currently assumes that failure by lateral torsional buckling or webpost buckling of beams does not govern. In some types of structural systems, such as cellular beams, these failure modes may govern and hence may present a more onerous failure mode than can be predicted by this method.
1.4 Outline of this document

In this document, Section 1 gives an introduction to the project and outlines the scope and objectives. Section 2 provides a literature review of vehicle fire incidents in carparking buildings. Section 0 lists the regulatory requirements of the NZBC and methods of assessment permitted by C/VM2.

The alternative assessment method for steel framed carparks is presented in Section 4, with a worked example of the method in Section 0. Section 6 lists the referenced documents.

0 lists the equations that are used in the proposed alternative assessment method.

0 outlines the spreadsheet calculations that have been used to demonstrate the feasibility of the proposed method.

0 provides additional calculation examples to consider different fire scenarios based on the structure described in Section 0.
2. LITERATURE REVIEW

2.1 Behaviour of fires in carparking buildings

Vehicle fires in carparking buildings primarily produce localised fires resulting in high temperatures in the close proximity area of the vehicle(s) on fire, but relatively temperatures further away from the fire. Based on statistics, full room involvement whereby all the vehicles in the carpark are burning simultaneously, is considered unlikely to occur.

2.2 Statistics of fires in carparking buildings

The following section summarises the statistics of fires in carparking buildings:

2.2.1 French Data

- Joyeux\textsuperscript{1} provides a comprehensive summary of studies and statistics into vehicle fires in carparks. The statistics are based vehicle fires reported by the Fire Brigade of Paris which involves 327 fire fighter intervention reports in underground carparks, and 78 reports in above-ground car parks.

![Figure 2-1: Distribution of numbers of vehicles involved in underground carpark fires](image)

---

Of the 327 reports in underground car parks, there were 158 car fires, involving 192 cars, of which the majority (98%) involved less than 4 cars (See Figure 2-1). There were two reported incidents which involved 7 cars, 1 involving 5 cars and 2 incidents what that involved 4 cars. One of the incidents which involved 7 cars was caused by arson. It is worth noting that 32% of the fires in car parks were not due to cars but was attributed to other combustibles such as paper, garbage and storage.

Figure 2-2 summarises the distribution of the categories of the reported car fires. It should be noted that this distribution is based on a smaller sample of vehicles (175 vehicles) as not all vehicles were determined after the fire. The categorisation of the vehicles in shown in Figure 2-3.
Figure 2-4: Distribution of numbers of vehicles involved in open deck carpark fires

- Figure 2-4 shows the distribution of the number of cars in car fire incidents in open deck car parks. 30% of the fires did not involve vehicles and were due to combustibles such as paper, garbage and stored materials. The remaining 70% of fires involved up to three cars, whereby 56% involved 1 car, 7% involved 2 cars and 8% involved 3 cars.
- There were 55 reported car fires in open deck car parks, involving a total of 72 cars. It was noted that the majority of the fires involved only 1 car and the maximum number is 3 cars.

The following figure (Figure 2-5) illustrates that fires in closed carparks (such as basement carparks) have a longer time to extinction. These reports show that fires in closed carparking structures last longer compared with open deck carparks. Fire is more likely to spread between cars compared with open deck car parks.

Figure 2-5: Fire Extinction Time for Open Deck and Underground Car Parks

2.2.2 New Zealand Data

A report by Li[6] stated that between 1995 and 2003, there were 93 incidents of vehicle fires in parking buildings in New Zealand of which 93 were single vehicle fires and three were multiple incident fires. The three reported multi-incident fires involved a combined total of eight vehicles. The findings of this review
were that fire can spread between vehicles especially in closed parking structures. All the fire tests reviewed in this study demonstrated the stability of the structure exposed to car fires.

Tohir and Spearpoint have complied the likelihood of car fires in carparks, based on research by Joyeux[1] and Li[6], which is summarised in Table 2-1 and Figure 2-6:

Table 2-1: Probability of car fire incidents Note 1

<table>
<thead>
<tr>
<th>Number of vehicles involved</th>
<th>Number of incidents</th>
<th>Proportion of incidents</th>
<th>Cumulative proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>344</td>
<td>0.858</td>
<td>0.858</td>
</tr>
<tr>
<td>2</td>
<td>27</td>
<td>0.067</td>
<td>0.925</td>
</tr>
<tr>
<td>3</td>
<td>21</td>
<td>0.052</td>
<td>0.977</td>
</tr>
<tr>
<td>4</td>
<td>4</td>
<td>0.010</td>
<td>0.987</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>0.007</td>
<td>0.994</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0.000</td>
<td>0.994</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>0.005</td>
<td>0.999</td>
</tr>
<tr>
<td>&gt; 7</td>
<td>0</td>
<td>0</td>
<td>0.999</td>
</tr>
</tbody>
</table>

Note 1: It should be noted that the reported incidents do not differentiate whether or not sprinklers were installed in the buildings.

Figure 2-6: Summary of probability and cumulative probability of vehicle fires involved

2.2.3 US Data

In a US study[8] of more than 400 parking garage fires, it was found that 80% of the fire incidents involved vehicle fires, and the remainder did not involve vehicles, caused by malfunction of equipment such as elevators, generators etc and by human error. It was reported that 7% of the fire incidents involved multiple vehicle fires, with fire mainly occurring between two adjacent vehicles, with one case involving three cars and another involving four cars.
2.2.4 Efficacy of sprinklers

The tests undertaken by Bennetts\cite{bennetts12} have shown the efficacy of sprinklers in controlling car fires, which showed that with a functioning sprinkler system that operated automatically, there was no spread of fire from the originating car to adjacent cars.

Statistical analysis of sprinkler protection records in Australia and New Zealand between 1886 and 1986, has been undertaken by Marryatt\cite{marryatt13}. With regards to car park facilities\footnote{Comprising automobile garages and service showrooms, and garages (part of major occupancy).}, the statistics indicate that 100% of 96 fires were controlled by the successful operation of the installed sprinkler systems. The statistics indicate:

- 56% of fires were controlled by the activation of 1 sprinkler head;
- 76% of fires were controlled by the activation of 2 sprinkler heads;
- 84% of fires were controlled by the activation of 3 sprinkler heads;
- 89% of fires were controlled by the activation of 4 sprinkler heads; and
- 95% of fires were controlled by the activation of up to 7 sprinkler heads

In only two fires more than eight sprinkler heads were activated.
3. REGULATORY REQUIREMENTS AND OVERVIEW OF METHODS

3.1 Regulatory requirements
C/AS7 of the NZBC requires that carparking structures achieve:

- 60 minute FRR where sprinklers are not installed or
- 30 minute FRR where sprinklers are installed

For steel structures, a 30 or 60 minute FRR would require the structural framing elements to be protected with passive fire protection.

The objective of this document is to determine if an alternative method of assessment can be applied to demonstrate a reduced fire resistance rating, whilst satisfying the Performance Requirements of the NZBC.

3.2 Overview of methods
This section provides a discussion of the methods for modelling burnout fires that are permitted by C/VM2 (Clause 2.4). The applicability of these methods for representing carpark fires is discussed.

C/VM2 (Clause 2.4) permits three options for representing burnout design fires, which are:

Use a time equivalent formula to determine the equivalent fire severity ($t_e$) and to ensure that the Fire Resistance Rating (FRR) is equal to or greater than the $\geq t_e$. C/VM2 states that an equivalent fire severity of 20 minutes to be used if the calculated value is less.

This method assumes that a compartment has reached flashover and assumes that the fuel load is uniformly distributed across the floor plate of the fire compartment. The method determines the equivalent fire severity under exposure to the ISO standard fire. This method does not consider the highly localised nature of the fire and fire load within the compartment. The conventional application of the time equivalence method is to treat an entire carpark floor as the fire compartment on the assumption that the compartment can reached the flashover conditions as assumed by the time equivalence method. This approach would produce an average fire severity across the fire compartment, but may not adequately consider the high fire severity in the area of fire involvement, but low severities in the far-field region (areas not involved in the fire). It is noted that a study by Spearpoint et al[16] have suggested that a reduced FLED of 260MJ/m$^2$ could be applied as an appropriate design FLED for such situations.

Use a parametric time versus gas temperature formula to calculate the thermal boundary conditions (time/temperature) for input to a structural response model.

The Parametric Fire method assumes a post flashover fire within the fire compartment. The conventional application of the Parametric Fire method is to treat an entire carpark floor as the fire compartment. With this approach it can result in lower temperatures and / or shorter durations than what could be typically expected in a highly localised real car fire. Similarly, zone models (OZONE), whilst being suitable for predicting far-field temperatures in closed carparks, can underpredict the localised temperatures[1] and higher fire severities in the near field region.

The parametric fire, zone models and the time equivalence method tend to predict low severities (fire temperatures and durations) because it uses an average and lower fuel load to represent the fuel load in carpark. The lower fuel load is attributed to the absence of fuel load in the driveway areas. Conversely, the use of an average design fuel load means that it does not account for the higher and localised fuel loads where the vehicles are located.
Construct a HRR versus time structural design fire as described in Paragraph 2.3.3 of C/VM2. Then, taking into account the ventilation conditions, use a fire model or energy conservation equations to determine suitable thermal boundary conditions (time/temperature/flux) for input to a structural response model.

This method is considered the most appropriate for representing the thermal exposure of a carpark structure to a localised vehicle fire. The proposed method is based on the principles of C/VM2 Clause 2.4(c), by using a heat-release rate curve to represent an isolated fire. This is described in Section 4.
This section presents an alternative assessment method to determine the structural adequacy of unprotected steel elements under realistic fires in carparks. The method will consider the localised nature of vehicle fires, instead of full involvement of an entire fire compartment under exposure to the AS1530.4 furnace fire, as assumed in the time equivalence method. This proposed method proposes to utilise realistic design fires based on experimental data of vehicle fire tests.

It will also determine the realistic load capacities of structural elements as a function of time.

4.1 Procedure of proposed method
The steps of the proposed method are:

1. Define the Design Fire based on heat release rates of real car fire tests in carparks.
2. Determine the height of the flames from the design fire.
3. Calculate the temperatures of the steel beams as a function of time, based on the design fire in Step 1.
4. Calculate the strength of the steel beam as a function of time.

4.2 Step 1: Definition of design fire
The design fire for carpark structures is proposed to be based on treating each vehicle fire as a localised fire which forms an axisymmetric fire plume.

- For a sprinklered carpark (open deck or enclosed carpark): One Class 3 vehicle on fire is proposed for the design fire.

The proposed design HRR-time is shown in Figure 4-1. The basis of this HRR curve is from calorimeter fire tests undertaken in Europe in the 1990’s, based on a Class 3 car (Peugeot 406, Volkswagen Passat) which corresponds to a fuel load of about 9500MJ in the car. Compared with the data of reported vehicle fires in France and New Zealand, a single car fire has an 85 percent probability of occurrence. In addition, the studies by Bennetts[12] have shown that with a functioning sprinkler system, fire spread to other cars is mitigated. Compared with HRR data presented by Tohir et al[3] for a range of various vehicle fires, the HRR proposed by Figure 4-1 is considered appropriate as it captures the maximum peak heat release and fire duration of most of the tested vehicles.

- For an unsprinklered carpark (open deck or enclosed carpark): Three Class 3 vehicles and one utility vehicle on fire are proposed for the design fire (See Appendix C.1 for HRR definition for utility vehicle).

To consider multiple vehicle fires, the heat release rate of each vehicle is considered separately. Figure 4-2 shows the HRR curves for three separate car fires. The HRR curve for the first vehicle is identical to that of the one-car fire scenario. The growth in the HRR of the second and the third vehicles occur 12 minutes after the start of the first and second vehicles respectively. The HRR of the second and third vehicles are based on experimental data of fire tests in Europe[14]. A significant amount of research on structural fire behaviour of steel framed carparks[11] have been subsequently based on the use of the HRR curves shown in Figure 4-1 and Figure 4-2.
Figure 4-1: Design HRR curve for a single Class 3 car fire$^{[1]}$ (Refer to Table 5.1 for tabulated values)

Figure 4-2: Separate HRR curves for three Class 3 cars
A utility vehicle fire is considered. The design heat release rate (HRR) for each of the single car fire scenario is shown in Figure 7 1, which is a fast growing fire that ramps up to 17MW in the first 3 minutes and then decays form 17 MW to zero between the 17th minute and the 25th minute. The HRR is based on information proposed by the ISO/TC 92 Technical Committee[15].

4.3 Step 2: Flame height check

The height of the flame of each plume is calculated from the equation below. The calculated flame height is compared with the clear height to the structure (Slab and beams) above the fire source, to determine if the fire plume will impinge onto the structural elements.

For an axisymmetric plume, the flame height, \( L_f \) for a localised fire is shown below:

\[
L_f = -1.02D + 0.0148Q^{2/5} \quad (m)
\]

Equation 4-1

Where:
- \( D \) is the diameter of the fire (m) \((D < 10m)\)
- \( Q \) is the rate of heat release of the fire (W) \((Q < 50MW)\)

4.4 Step 3: Calculation of Plume and Beam Temperatures

Based on the calculation in Step 2 above, the temperatures of the structure above can be calculated:

- If the structure is not engulfed by flames, the temperatures of the structural member can be represented by the temperature along the axis of the plume, as per the method in Section 4.4.1 below.
- If the plume impinges onto the structure, refer to Section 4.4.2 to determine the heat flux onto the structure. The heat flux shall be determined at the underside of the beam, for conservatism.

4.4.1 Without flame impingement

The temperature in the plume \( \Theta(z) \) along the symmetrical vertical flame axis is given by:

\[
\Theta(z) = 20 + 0.25Q_c^{2/3}(z - z_0)^{6/3} \leq 900^\circ C \quad (^\circ C)
\]

Equation 4-2

Where the virtual origin \( z_0 \) of the axis if given by:
- $z_0 = -1.02D + 0.00524Q^{0.5}$ (m) (Where $Q \leq 50$MW)
- $Q_c$ = the convective part of the rate of heat release (W) with $Q_c = 0.8Q$
- $z =$ the height along the flame axis from the base of the fire, which is the base of the vehicle and that may be slightly elevated from the ground (m)

**Figure 4-4: Localised fire – for flames which do not impinge on the slab/beam above**

### 4.4.2 Flame impingement equations

If the flames impinge onto the slab / beams above, Eurocode 1-2 provides equations for calculating the heat flux on the underside of a slab/beam.

These equations are based on the Hasemi model. The equations consider the height of the ceiling above the fire, the heat release rate (HRR) and radial distance from the fire.

**Figure 4-5: Localised fire – for flames which impinge on the slab /beam above**

The heat flux, $q''$ (kW), received at the underside of a beam/slab is:

- $q'' = 100$ if $y \leq 0.30$
- $q'' = 136.3 - 121y$ if $0.3 < y < 1.0$
- $q'' = 15y^{3.7}$ if $y \geq 1.0$

where:

- $y = \frac{r + H + z}{L_H + H + z}$
- $Q_{h} = \frac{q}{1.11 \times 10^{6} \cdot H^{2.5}}$
- $Q_{b} = \frac{q}{1.11 \times 10^{6} \cdot D^{2.5}}$
- $L_H = horizontal$ $flame$ $length = 2.9H(Q_{h})^{0.33} - H$
To consider the effect of multiple vehicles, the heat release rate of each vehicle will need to be determined and the heat flux for each vehicle (at different radial distances to the subject element) will need to be considered and added, as shown below:

\[ q'' = q''_{car1}(r) + q''_{car2}(r) + q''_{car3}(r) + \ldots \]  

Equation 4-3

The net heat flux is calculated as follows:

\[ h_{net} = q'' - \alpha_c(\theta_m - 20) - \Phi \varepsilon_m \varepsilon_f \sigma[\theta_m + 273]^4 - 293^4] \]  

where:
- \( \alpha_c = 35 \) for parametric fires (W/m\(^2\)K)
- \( \varepsilon_m \) = surface emissivity of the member = 0.7 from EC3-1-2:2005
- \( \varepsilon_f = \) fire emissivity = 1.0 from EC3-1-2:2005
- \( \sigma = \) the Stephen-Boltzmann constant = 5.67 x 10\(^{-8}\) (W/m\(^2\)K\(^4\)) from EC3-1-2:2005
- \( \Phi = \) configuration or view factor = 1.0

The temperatures of the structural elements can be calculated using three methods:

1. Estimation based on an approximation of the gas temperatures
2. An iterative spreadsheet method assuming a homogenous temperature across the steel section, or
3. A finite element method to calculate the temperature distribution in the steel section. This is not covered in this document.

Note that method 1 will result in more conservative results compared with the other methods.

For method 1, the following maximum temperatures may be used to approximate the temperatures of the different components\(^2\):

\begin{align*}
T_{\text{bottom flange}} & = 0.95 T_{\text{fire}} \\
T_{\text{web}} & = 0.95 T_{\text{fire}} \\
T_{\text{top flange}} & = 1.0 T_{\text{fire}} - 150^\circ \text{C when at least 85\% of a flange is covered} \\
T_{\text{top flange}} & = 0.95T_{\text{fire}} \text{ when less than 85\% of a flange is covered}
\end{align*}

For method 2, with the iterative spreadsheet method, the following equation is to be used\(^3\):

\[ \Delta \theta_{at} = \frac{\alpha_{sh}}{\alpha_{pa}} \frac{\lambda_{w/w}}{h_{net} \Delta t} \]  

Equation 4-5

\(^2\) Now included in AS/NZS 2327:2017 Composite structures - Composite steel-concrete construction in buildings
Where:

- $\Delta \theta_{a,t} = \text{Increase of temperature in an unprotected steel member during a time interval } \Delta t$
- $\Delta t = \text{Time interval (recommended not greater than 60s)}$
- $k_{sh} = \text{Shadow factor} = 0.9 \left[ \frac{A_m/V}{b} / \frac{A_m/V}{b} \right]$
- $A_m/V = \text{Section factor for unprotected steel member (1/m) (not to be less than 10m}^{-1})$
- $[A_m/V]_b = \text{Box value of the section factor}$
- $A_m = \text{Surface area of member per unit length (m}^2/m)$
- $V = \text{Volume of the member per unit length (m}^3/m)$
- $c_a = \text{Specific heat of steel (m}^3/m)$
- $h_{net} = \text{design value of the net heat flux per unit area (W/m}^2)$

4.5 Step 4: Calculation of member capacity

The calculated steel temperatures are used to determine the member capacities. The residual capacities (Moment, shear and/or axial) of the elements can be determined (with input from structural engineers). The residual capacities of the elements ($R_F$) are compared with the fire limit state actions ($U^*_{fire}$) so that:

$$R_F \geq U^*_{fire}$$

- For example: A check of the flexural capacity in the fire limit state ($M_i$) against the applied moments in the fire limit state ($M_{i\text{Fire}}$) would be: $M_i \geq M_{i\text{Fire}}$.
- If the capacity ($R_F$) is greater than the action ($U^*_{fire}$), then the structural element is considered to be able to resist the fire limit state actions.
- If the capacity ($R_F$) is less than the action ($U^*_{fire}$), then the size and/or strength of the structural element would need to be increased. Alternatively, other load resisting mechanisms (load redistribution or two-way slab behaviour via tensile membrane action) would need to be considered.
- Load redistribution to other adjacent beams is permitted provided that the composite floor system has been detailed and reinforcing mesh and bars lapped appropriately; the ability of the loads to transfer to adjacent beams will need to be confirmed with the design structural engineer.
- This method of comparison is based on the assumption of one-way action and does not consider other mechanisms such as two-way action (tensile membrane action) that can be expected in a real multi-bay steel framed carpark structure. Two-way action in the slab is not covered in this document but can be covered by a more advanced analysis method, e.g.: finite element analysis.
- The proposed method assumes that composite action is provided between the beams and slab. The method does not allow for failure by lateral torsional buckling of the beams.

4.6 Locations of Design Checks

For composite beams in a carpark, the following checks should be carried out:

1. The bending and shear checks of the beams at the location of highest likely fuel loads. This is likely to be the carparking bays.
2. The bending capacity check of the beam at the location of highest bending moment (E.g.: midspan of the beam).
3. The residual shear strength of the beam at the location of highest shear force.
- Checks for Items 2 and 3 are undertaken because Item 1 (the area of highest likely fuel load) may not have the highest moments and shear forces on the beams.
The checks should be conducted for a typical structural configuration and also in non-typical structural configurations.

A sensitivity check for a single vehicle fire should also be undertaken to consider a fire along the driveways, if not already covered by Items 1-3.

This is depicted schematically in Figure 4-6 and Figure 4-7 for a sprinklered and unsprinklered carparks respectively.

**Figure 4-6: Fire locations for sprinklered carpark**
- Scenario S1: One car fire under a beam
- Scenario S2: One car on fire along the driveway under a beam
- Scenario S3: One car fire at the perimeter / corner of the carpark

**Figure 4-7: Fire locations for unsprinklered carpark**
- Scenario U1: Three cars on fire under a beam(s)
- Scenario U2: Two cars on fire along the driveway under a beam(s)
- Scenario U3: Three cars on fire at the perimeter / corner of the carpark
- Scenario U4: One utility vehicle under a beam
- Scenario U5: One utility vehicle on fire along driveway under a beam
- Scenario U6: One utility vehicle on fire at the perimeter / corner of the carpark
5. WORKED EXAMPLE

A worked example is described below to explain how the proposed method works:

5.1 Description of Structure

The following structural configuration is shown as an example of a typical multi-storey carparking building in New Zealand. The carpark is fully sprinkler protected. The structure has a uniform grillage of beams with long span beams (17m) which span across the carparking spaces located on the two sides of the driveway. The inter-storey height is 3.0m.

The columns are require to achieve the required Fire Resistance Rating either by passive fire protection, or if unprotected, by its inherent fire resistance. Beam connections are to be protected.

Calculations are undertaken to determine if the primary beams (250UC72.9 beams) and the secondary beams (600WB190 beams) can be unprotected or not.

Figure 5-1: Plan view showing part of a typical steel framed carpark

5.2 Structural sections

The long span secondary beams are spaced at 3.8m centres. The primary beams which run left to right are 250UC72.9 sections. The beams support a 150mm thick Comflor 80 composite steel-concrete slab. The beams are assumed to be simply supported. The details of the structural elements are as follows:

<table>
<thead>
<tr>
<th>Section size</th>
<th>Depth (mm)</th>
<th>Flange width (mm)</th>
<th>Flange thickness (mm)</th>
<th>Web thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Secondary beams</td>
<td>600</td>
<td>350</td>
<td>25</td>
<td>12</td>
</tr>
<tr>
<td>600WB190</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Material properties

- Structural Steel: 300MPa
- Reinforcing: Grade 500 E Mesh
- Concrete compressive strength: 30MPa

### Description of Fire Scenarios

Two separate single car fire scenarios are considered, as shown below in Figure 5-2. The fire scenarios are described as:

- **Scenario 1:** Fire in a single parked car. A single car fire would represent a credible design fire, considering the benefits of sprinklers to prevent spread of fire to adjacent vehicles. The parked car is located near the secondary beam support where the beam shear forces are high. For the worst case heating condition, the beam is assumed to be located directly beneath the beam (rather than offset from the beam centreline).

- **Scenario 2:** Single car along the driveway directly beneath a beam. This is a less likely scenario because the driveway is typically not occupied, but is considered nevertheless. A likely vehicle fire on the driveway could be due to an accident on the driveway. This location has the highest bending moment on the beam.

---

### Table: Section Sizes and Dimensions

<table>
<thead>
<tr>
<th>Section size</th>
<th>Depth (mm)</th>
<th>Flange width (mm)</th>
<th>Flange thickness (mm)</th>
<th>Web thickness (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary beams - 250UC72.9</td>
<td>260</td>
<td>256</td>
<td>17.3</td>
<td>10.5</td>
</tr>
<tr>
<td>Columns - 310UC158</td>
<td>327</td>
<td>311</td>
<td>25</td>
<td>15.7</td>
</tr>
</tbody>
</table>

The reinforcing mesh consists of SE92 Grade 500 E Mesh (9mm diameter at 200mm grid spacing).
5.5 Scenario 1

5.5.1 Step 1: Design Fire

The design heat release rate for each of the single car fire scenario is shown in Figure 4-1.

![Heat Release Rate Graph](image)

**Figure 5-3: HRR vs time for a single vehicle fire**

<table>
<thead>
<tr>
<th>Time (Minutes)</th>
<th>HRR (MW)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>1.4</td>
</tr>
<tr>
<td>16</td>
<td>1.4</td>
</tr>
<tr>
<td>24</td>
<td>5.5</td>
</tr>
<tr>
<td>25</td>
<td>8.3</td>
</tr>
<tr>
<td>27</td>
<td>4.5</td>
</tr>
<tr>
<td>38</td>
<td>1</td>
</tr>
<tr>
<td>70</td>
<td>0</td>
</tr>
</tbody>
</table>

This scenario considers a fire of a single parked car, as shown in Figure 5-2 and Figure 5-4. The length of the vehicle parking space is assumed to be 5m long and the fire is assumed to be located at the middle of the parking space. Therefore, the fire location is assumed to be located 2.5m from one end of the beam, as shown in Figure 5-4. The heat release rate of the fire is assumed to follow that shown in Figure 5-3.

![Schematic Diagram](image)

**Figure 5-4: Schematic showing side elevation and location of car fire.**
5.5.2 Step 2: Check if the flame height will impinge on the underside of the beam

Using Equation 4-1, the flame height, \( L_f \) =

\[
L_f = -1.02D + 0.0148Q^{2/5}
\]

Assuming:
Heat flux per unit area of the fire = 1MW/m²

Note:
Heat flux per unit area (HRR PUA) is based on the peak heat HRR (8.3MW) divided by 75% of a typical car parking space (75% of 2m x 5m) as from observations the entire area of carparking spot is not involved in the fire at the peak heat release rate

Therefore for the maximum HRR (8.3MW),

\[
\text{Equivalent circular burning area of the fire} = \frac{\text{Maximum HRR}}{\text{HRR PUA}} = \frac{(8.3 \text{MW})}{1\text{MW/m}^2} = 8.3\text{m}^2
\]

\[
\text{Diameter of fire, } D = \left[\frac{(8.3\text{m}^2) \times 4 / 3.142}{0.5}\right] = 3.25\text{m}
\]

\[
\text{Flame Length, } L_f = -1.02D + 0.0148Q^{2/5}
\]

\[
= -1.02 \times 3.25\text{m} + 0.0148 \times (8,300,000\text{ W})^{2/5}
\]

\[
= 5.35\text{m}
\]

As \( L_f \) (5.35m) is greater than the height to underside of slab above (2.85m), there will be flame impingement on the beams and underside of the slab.

5.5.3 Step 3: Calculation of Plume and Beam Temperatures

5.5.3.1 Beam 600WB 190

The 600WB190 beam directly above the fire is consider here. The radial distance of the beam from the fire, \( r \), is zero. Based on equations in Section 4.4.2, the temperatures of the flux onto the steel beam can be calculated.

An example of the calculation, based on the maximum flux (corresponding to the peak heat heat release rate of 8.3MW) is shown below:

\[
H_s = \text{Height of lower flange of beam from the floor} = 2.85\text{m} - 0.6\text{m} = 2.25\text{m}
\]

\[
H_t = \text{Height of fire source from the floor} = 0.3\text{m}
\]

\[
H = H_s - H_t = 2.25\text{m} - 0.3\text{m} = 1.95\text{m}
\]

\[
Q_{ib} = \frac{Q}{1.11 \times 10^6 \times 1.95^{2.5}} = 8.3 \times 10^6 / (1.11 \times 10^6 \times 1.95^{2.5}) = 1.4
\]

\[
Q_{ib} = \frac{Q}{1.11 \times 10^6 \times 3.25^{2.5}} = 8.3 \times 10^6 / (1.11 \times 10^6 \times 3.25^{2.5}) = 0.39
\]
\[ z' = 2AD \left( Q_D^{2/3} - Q_D^{2/3} \right) = 1.19 \]

\[ L_H = 2.9H(Q_H)^{0.33} - H = 2.9 \times 1.95 \times 1.4^{0.33} - 1.95 = 4.38m \]

\[ y = \frac{r+H+z'}{L_H+H+z'} = \frac{0+1.95+1.19}{4.38+1.95+1.19} = 0.42 \]

\[ q^* = 136300 - 121000 \times 0.42 = 85,776W/m^2 = 85.8kW/m^2 \]

Solving Equation 4-4 by setting \( \dot{h}_{net} = 0 \) and assuming a steady state condition the steel temperature is equal to the flame temperature \((\theta_m = \theta_f)\), the flame temperature \((\theta_f)\) can be solved. To do this, a spreadsheet analysis utilising the ‘Goal-seek’ function is applied to solve for the flame temperature.

Applying an incremental method of the analysis, the flame temperature impinging on the steel can solved for the whole fire duration. This is applied in a spreadsheet analysis.

![Graph](image)

**Figure 5-5: Calculated fire temperature above at the soffit of beam above the fire.**

The temperature of the steel beams can be calculated iteratively using Equation 4-5. For the 600WB190 beam, the heat transfer calculation is undertaken for the top flange, bottom flange and web whereby the section factor for each element can be determined separately. The inputs for the heat transfer calculations are as follows:

<table>
<thead>
<tr>
<th>Structural element</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Element description</td>
<td>bottom flange</td>
<td>top flange</td>
<td>web</td>
</tr>
<tr>
<td>( H_{ef}/A ) ((m^2))</td>
<td>80</td>
<td>40</td>
<td>167</td>
</tr>
<tr>
<td>Shadow factor</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Effective View factor</td>
<td>85.7</td>
<td>45.7</td>
<td>186.7</td>
</tr>
</tbody>
</table>
The corresponding steel temperatures are shown in Figure 5-6.

### 5.5.3.2 Beam 250UC72.9

The 250UC72.9 primary beam is located 2.5m from the centreline axis of the fire. The radial distance of the beam from the fire, $r$, is 2.5m. Based on equations in Section 4.4.2, the flame temperatures and flux onto the steel beam can be calculated.
The temperature of the steel beams can be calculated incrementally using Equation 4-5. For the 250UC72.9 beam, the heat transfer calculation is undertaken for the top flange, bottom flange and web whereby the section factor for each element can be determined separately. The inputs for the heat transfer calculations are as follows:

<table>
<thead>
<tr>
<th>Structural element</th>
<th>1</th>
<th>2</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Element description</td>
<td>bottom flange</td>
<td>top flange</td>
<td>web</td>
</tr>
<tr>
<td>$H_p/A$ (m$^{-1}$)</td>
<td>149</td>
<td>78</td>
<td>233</td>
</tr>
<tr>
<td>Shadow factor</td>
<td>1.0</td>
<td>1.0</td>
<td>1.0</td>
</tr>
<tr>
<td>Effective View factor</td>
<td>149</td>
<td>78</td>
<td>233</td>
</tr>
</tbody>
</table>

Convection coefficient $\alpha_c = 35$ W/m$^2$K

Radiation coefficients:
- Fire emissivity, $\varepsilon_f = 1.00$  
  (EC3-1-2:2005)
- Surface emissivity of member, $\varepsilon_m = 0.70$  
  (EC3-1-2:2005)
- Configuration factor, $\phi = 1.00$
- Convective fraction = 0.80

The corresponding steel temperatures are shown in Figure 5-9.
Figure 5-9: Calculated steel temperatures of 250UC72.9 beam located r=2.5m from the fire.

5.5.4 Structural Calculations

5.5.4.1 Beam 600WB 190

Flexural capacity check

Using the temperatures calculated in Figure 5-6, the flexural capacity of the beam can be calculated and compared with the applied bending moment.

- The flexural capacity of the beam can be calculated based on the method by Eurocode 4-1-2\(^{(10)}\). The equations are presented in Appendix A.1.
- Loading
  - Dead load, DL = Slab + metal deck + SDL = 2.54 + 0.15 + 0.5 = 3.19kPa
  - Live load, LL = 2.5 kPa
  - Fire limit state load = DL + 0.4 LL = 4.19kPa
  - Tributary width = 3.8m
  - W = UDL load + SW of beam = 4.19kPa x 3.8 m + 2kN/m = 17.9kN/m
- The applied bending moment is calculated at 2.5m from the end support of the beam (directly above the fire), which is calculated as:

$$M_{fire, 2.5m} = W \cdot x/2 \cdot (L-x) = 325kNm$$

Where:

- L = full length of beam = 17m
- x = distance at point of interest from end support = 2.5m

The variation of the flexural capacity of the composite beam as a function of temperature is summarised in Figure 5-10. The yield strength of the steel varies as a function of temperature, based on the material
properties from Eurocode 3-1-2\cite{9}. Appendix A.3 shows the residual yield strength of the steel that are used in the calculations.

The minimum flexural capacity of the beam is 454\(\text{kNm}\), which is greater than the applied bending moment of 325\(\text{kNm}\). Therefore, the beam is considered to have sufficient flexural capacity at this location.

![Figure 5-10: Comparison of flexural capacity and applied bending moment for 600WB190 beam](image)

**Shear capacity check**

Similarly, based on the steel temperatures calculated in Figure 5-6, the shear capacity of the beam can be calculated and compared with the applied shear force.

- The shear capacity of the beam can be calculated based on the method by Eurocode 4-1-2\cite{10}. The equations are presented in Appendix A.2.
- The applied shear force is calculated at 2.5m from the end support of the beam, which is calculated as:

\[
V_{\text{fire,2.5m}} = W.L/2 - W.x = 108\text{kN}
\]

Where:

\[
L = \text{full length of beam} = 17\text{m}
\]

\[
x = \text{distance at point of interest from end support} = 2.5\text{m}
\]

The variation of the shear capacity of the composite beam as a function of temperature is summarised in Figure 5-11. The minimum shear capacity of the beam is 183\(\text{kN}\), which is greater than the applied shear force of 108\(\text{kN}\). Therefore, the beam is considered to have sufficient shear capacity at this location.
5.5.4.2 Beam 250UC72.9

Flexural capacity check

Using the temperatures calculated in Figure 5-9, the flexural capacity of the beam can be calculated and compared with the applied bending moment.

- The flexural capacity of the beam can be calculated based on the method by Eurocode 4-1-2\textsuperscript{[10]}. The equations are presented in Appendix A.1.
- Surface Load = 4.19 kPa (See Section 5.5.4.1)
  - Tributary width = 3.8m
  - \( W = \text{UDL load on slab surface} + \text{SW of beam} = 4.19\text{kPa} \times 3.8 \text{ m} + 0.73\text{kN/m} = 16.7\text{kN/m} \)
- The applied bending moment is calculated at the worst location (at midspan) which is:

\[
M_{fire, midspan} = \frac{W.x}{2}(L-x) = 30\text{kNm}
\]

Where:

\[
L = \text{full length of beam} = 3.8\text{m}
\]
\[
x = \text{distance at point of interest from end support} = \frac{3.8\text{m}}{2} = 1.9\text{m}
\]

The variation of the flexural capacity of the composite beam as a function of temperature is summarised in Figure 5-12. The minimum flexural capacity of the beam is 582kNm, which is greater than the applied bending moment of 30kNm. Therefore, the beam is considered to have sufficient flexural capacity at this location.
Shear capacity check

Similarly, based on the steel temperatures calculated in Figure 5-6, the shear capacity of the beam can be calculated and compared with the applied shear force.

- The shear capacity of the beam can be calculated based on the method by Eurocode 4-1-2\textsuperscript{[10]}, The equations are presented in Appendix A.2.
- The applied shear force is calculated at 2.5m from the end support of the beam, which is calculated as:

\[
V_{\text{fire,} 2.5m} = \frac{W \cdot L}{2} - W \cdot x = 32kN
\]

Where:

\[
W = \text{Applied Load} / \text{m length} = 16.7kN/m
\]

\[
L = \text{full length of beam} = 3.8m
\]

\[
x = \text{distance at point of interest from end support} = 0m
\]

The variation of the shear capacity of the composite beam as a function of temperature is summarised in Figure 5-13. The minimum flexural capacity of the beam is 295kN, which is greater than the applied shear force of 32kN. Therefore, the beam is considered to have sufficient shear capacity at this location.
Figure 5-13: Comparison of shear capacity and applied shear force for Scenario 1.

5.5.5 Summary for Scenario 1

The analysis presented here has shown that the unprotected steel primary and secondary beams have sufficient flexural and shear capacity to resist the applied actions, when the structure is subjected to the Scenario 1 car fire.
5.6 Scenario 2:
This scenario considers a single car fire located along the driveway, at midspan of the secondary long span beam, as shown in Figure 5-2 and Figure 5-14. This location corresponds to the highest bending moment along the length of the beam. The heat release rate of the fire is based on that shown in Figure 5-3.

Figure 5-14: Schematic showing side elevation and location of car fire.

5.6.1 Temperature calculations for 600WB190 secondary beam (Directly above fire)
The temperatures of the beam directly above the fire have been previously calculated, as shown in Section 5.5.3 and Figure 5-6.

5.6.2 Structural Calculations
Flexural capacity check
The flexural capacity of the beam is similar to that of Scenario 1. However, the applied bending moment at midspan is greater compared with that of Scenario 1. The applied bending moment is calculated at midspan (8.5m from the end support of the beam), which is calculated as:

\[ M_{\text{fire, midspan}} = \frac{W \cdot x}{2} \cdot (L-x) = 647 \text{kNm} \]

Where:

\[ W = \text{Applied Load / m length} = 17.9 \text{kN/m} \]
\[ L = \text{full length of beam} = 17 \text{m} \]
\[ x = \text{distance at point of interest from end support} = 8.5 \text{m} \]

The variation of the flexural capacity of the composite beam as a function of temperature is summarised in Figure 5-15. The applied bending moment (647kNm) is greater flexural capacity of the beam. As shown in Figure 5-15, the flexural capacity of the beam equals the applied moment at approximately 22 minutes; a plastic hinge forms at the beam midspan at this time.
This means that the beam would not be able to support the applied moments. However, it must be noted that the fire is localised. Therefore, the flexural capacity of the two adjacent beams on either side of the heated beam is considered for resisting the moments that cannot be resisted by the main heated beam. This is shown schematically in Figure 5-16, whereby the adjacent beams are marked in blue.

The temperatures of the adjacent beams is checked using the same procedure as outlined in Section 4.4, but using R (radius) as 3.8m.
Figure 5-17: Calculated steel temperatures of beam located radially 3.8m from fire.

The calculated steel temperatures of the top flange, web and bottom flange of the adjacent beams located 3.8m from the plume centreline are shown in Figure 5-17.

Figure 5-18: Comparison of flexural capacity and applied bending moment for each secondary beam located 3.8m from the fire location.

The calculated bending capacity of each of the adjacent beams is shown in Figure 5-18. The flexural capacity of the beam does not reduce with time. This is because the steel temperatures are relatively low (Under 250°C) and the yield strength of the steel has not reduced. The applied moment on each of the beams increase after 20 minutes due to the moments that are transferred from the beam above the fire that is directly heated above the fire. Therefore, the analysis shows that even if the beam forms a plastic hinge, the adjacent beams can resist the applied loads.

Shear capacity check

Similarly, based on the steel temperatures calculated in Figure 5-6, the shear capacity of the beam can be calculated and compared with the applied shear force.

- The shear capacity of the beam can be calculated based on the method by Eurocode 4-1-2\textsuperscript{[10]}. The equations are presented in Appendix A.2.
The applied shear force at midspan of the beam, which is calculated as:

\[ V_{\text{fire, 2.5m}} = \frac{W.L}{2} - W.x = 0 \text{KNm} \]

Where:

\[ W = \text{Applied Load / m length} = 17.9 \text{kN/m} \]

\[ L = \text{full length of beam} = 17 \text{m} \]

\[ X = \text{distance at point of interest from end support} = 8.5 \text{m} \]

The variation of the shear capacity of the composite beam as a function of temperature is summarised in Figure 5-19. The minimum flexural capacity of the beam is 147kN, which is greater than the applied shear force. Therefore, the beam is considered to have sufficient shear capacity at this location.

![Graph showing shear capacity and applied shear force](image)

**Figure 5-19: Comparison of shear capacity and applied shear force for Scenario 2 (Main beam directly above the fire).**

### 5.6.3 Summary for Scenario 2

The analysis presented here has shown that the unprotected steel secondary beams have sufficient flexural and shear capacity to resist the applied actions, when the structure is subjected to a realistic vehicle fire for Scenario 2.
6. SUMMARY

This document has proposed an alternative design method for assessing the fire resistance of steel beams under localised fires, for application in carparking buildings. The proposed method is dependent on the vehicle design fires, beam spacings and the distance of the fire source to the beams.

Based on the worked example in this document for a realistic vehicle fire, the analysis has shown that the typical primary and secondary beams of the structure can be unprotected. The proposed method shows that realistic vehicle fires in carparking buildings will have only local impact onto structural beams. Beams that are located further away from the fire source have been shown to have high residual strength as the temperature impact is lower.
7. REFERENCES


2. BSI (2009), Eurocode 1 – “Actions on structures general actions - actions on structures exposed to fire”, BS EN 1991-1-2, British Standards Institution, 2002


4. Tohir, M., Spearpoint, M. Simplified approach to predict heat release rate curves from multiple vehicle fires in car parking buildings. 3rd International Conference on Fire in Vehicles (FIVE), Berlin, Germany; 10/2014


APPENDIX A  STRUCTURAL CAPACITY EQUATIONS

A.1 Beam bending capacity

The bending capacity calculated based on Eurocode 4-1-2[10]. The Eurocode method is proposed to be used as it currently has the most comprehensive method of analysis composite construction including the effects of fire.3

\[
T^+ = \left[ f_{ay\theta_1}(b_1 e_1) + f_{ay\theta_2 w}(h_w e_w) + f_{ay\theta_2}(b_2 e_2) \right] / \gamma_{M,fla} \tag{E.1}
\]

\[
y_T = \left[ f_{ay\theta_1}(b_1)(e_1^2/2) + f_{ay\theta_2 w}(h_w e_w)(e_1 + h_w/2) + f_{ay\theta_2}(b_2 e_2)(h - e_3/2) \right] / (T^+ \gamma_{M,fla}) \tag{E.2}
\]

with \( f_{ay\theta} \) the maximum stress level according to 3.2.1 at temperature \( \theta \) defined following 4.3.4.2.2.

(2) In a simply supported beam, the value of the tensile force \( T^+ \) obtained from (1) is limited by:

\[
T^+ \leq N P_{f,Rd} \tag{E.3}
\]

where:

\( N \) is the smaller number of shear connectors related to any critical length of the beam and \( P_{f,Rd} \) is the design shear resistance in the fire situation of a shear connector according to 4.3.4.2.5.

NOTE: The critical lengths are defined by the end supports and the cross-section of maximum bending moment.

(3) The thickness of the compressive zone \( h_u \) is determined from:

\[
h_u = T^+ / \left( b_{eff} f_c / \gamma_{M,flc} \right) \tag{E.4}
\]

where \( b_{eff} \) is the effective width according to 5.4.1.2 of EN 1994-1-1, and \( f_c \) the compressive strength of concrete at room temperature.

---

3 This report was prepared prior to publication of AS/NZS 2327:2017 Composite structures - Composite steel-concrete construction in buildings. Users are encouraged to refer to this standard for structural calculations.
(4) Two situations may occur:

\[(h_c - h_u) \geq h_{cr}\] with \(h_u\) is the depth \(x\) according to Table D.5 corresponding to a concrete temperature below 250°C. In that situation the value of \(h_u\) according to equation (E.4) applies.

or \((h_c - h_u) < h_{cr}\); some layers of the compressive zone of concrete are at a temperature higher than 250°C. In this respect, a decrease of the compressive strength of concrete may be considered according to 3.2.2. The \(h_u\) value may be determined by iteration varying the index "n" and assuming on the basis of Table D.5 an average temperature for every slice of 10 mm thickness, such as:

\[T^+ = F = \left[ \left(h_c - h_{cr}\right) f_c + \sum_{i=2}^{\text{num} } \left(10 h_{cr} f_{ck} + \left(h_{cr} h_{eff} f_{ck}\right) \right) \gamma_{M,fl,c} \right] / \gamma_{M,fl,c} \]  

(E.5)

where:

\[h_u = \left(h_c - h_{cr}\right) + 10(n-2) + h_{cr} \]  

[mm]

\(n\) is the total number of concrete layers in compression, including the top concrete layer \(h_c - h_{cr}\) with a temperature below 250°C.

(5) The point of application of this compression force is obtained from

\[y_F = y + \left(h_u / 2\right) \]  

(E.6)

and the sagging moment resistance is

\[M_{fl,Rd} = T^+ \left(y_F - y_F\right) \]  

(E.7)

with \(T^+\), the tensile force given by the value of (E.5) while taking account of (E.3).

(6) This calculation model may be used for a composite slab with a profiled steel sheet, provided in (3) and (4), \(h_c\) is replaced by \(h_{eff}\) as defined in (1) of D.4 and \(h_u\) is limited by \(h_f\) as defined in Figures 4.1 and 4.2.

(7) This calculation model established in connection to 4.3.4.2.4, may be used for the critical temperature model of 4.3.4.2.3 by assuming that \(\theta_j = \theta_{cr} = \theta_{cr}\).

(8) A similar approach may be used if the neutral axis is not inside the concrete slab but in the steel beam.
A.2 Shear Capacity Calculations

The shear capacity is estimated based on Eurocode 3-1-1 as:

\[ V_{pl,le} = \frac{A_v \left( \frac{f_y}{\sqrt{3}} \right)}{\gamma_M} \]

Where

\[ A_v = \text{shear area} = A - 2b.t_t + (t_w + 2r)t_t \text{ for I beams} \]
\[ f_y = f_{y,0} = \text{yield strength of steel at elevated temperature} \]
\[ \gamma_M = \gamma_{M,0} = 1.0 \text{ for fire limit state} \]

A.3 Reduction factor of yield strength for hot rolled steel

The figure and table above show the reduction factors of the yield strength of steel as a function of temperature that are used in the calculations. These are based on Eurocode 3-1-2\(^9\).
APPENDIX B  EXAMPLE CALCULATIONS

The following shows an example spreadsheet calculation for heat transfer of the steel and the structural capacity calculations.
B.1 Heat transfer calculation
B.2 Structural capacity calculation

Table B.3: Temperature distribution in a solid slab of 160 mm thickness component of normal weight concrete and not insulated.

<table>
<thead>
<tr>
<th>Time (s)</th>
<th>Fire temp (°C)</th>
<th>Water temp (°C)</th>
<th>Fc (MPa)</th>
<th>fy (MPa)</th>
<th>Fy (MPa)</th>
<th>T (°C)</th>
<th>γm</th>
<th>γv</th>
<th>γv'</th>
<th>γv''</th>
<th>γv'''</th>
<th>Time (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>32.9</td>
</tr>
<tr>
<td>0.50</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>15.0</td>
</tr>
<tr>
<td>1.00</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>7.5</td>
</tr>
<tr>
<td>1.50</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>4.0</td>
</tr>
<tr>
<td>2.00</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>2.5</td>
</tr>
<tr>
<td>2.50</td>
<td>350.0</td>
<td>20</td>
<td>310</td>
<td>330</td>
<td>340</td>
<td>250</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>1.4</td>
</tr>
</tbody>
</table>

For the purpose of this calculation, it is assumed that the average temperature (T) is the same as the fire temperature (T_f) and the steel capacity (Fy) remains constant throughout the time.

The bending capacity of the beam is determined by:

\[ \text{Bending capacity} = \text{Flange capacity} + \text{Web capacity} \]
APPENDIX C  ADDITIONAL CALCULATIONS

This section presents additional fire scenarios to the worked example in Section 0.

C.1 Utility Vehicle Fire

C.1.1 Step 1: Design Fire

A utility vehicle fire is considered. The design heat release rate (HRR) for each of the single car fire scenario is shown in Figure 7-1, which is a fast growing fire that ramps up to 17MW in the first 3 minutes and then decays from 17 MW to zero between the 17th minute and the 25th minute. The HRR is based on information proposed by the ISO/TC 92 Technical Committee[15].

![Figure 7-1: HRR for utility vehicle](image)

The utility vehicle is assumed to be located along the driveway, at midspan of the secondary long span beam, as shown in Figure 5-2. This location corresponds to the highest bending moment along the length of the beam.

C.1.2 Step 2: Check if the flame height will impinge on the underside of the beam

Using Equation 4-1, the flame height, \( L_f = \)

\[
L_f = -1.02D + 0.0148Q^{2/5}
\]

Assuming:

Heat flux per unit area of the fire = 1MW/m²

Therefore for the maximum HRR (17MW),

\[
\text{Equivalent circular burning area of the fire} = \frac{\text{Maximum HRR}}{\text{HRR PUA}} = \frac{17\text{MW}}{1\text{MW/m}^2} = 17\text{m}^2
\]

\[
\text{Diameter of fire, } D = [(17\text{m}^2) x 4 / 3.142]^{0.5} = 4.65\text{m}
\]
Flame Length, \( L_f \) = \[-1.02D + 0.0148Q^{2/5}\]
= \[-1.02 \times 4.65m + 0.0148 \times (17,000,000 W)^{2/5}\]
= 6.8m

As \( L_f \) (6.8m) is greater than the height to underside of slab above (2.85m), there will be flame impingement on the beams and underside of the slab.

C.1.3 Step 3: Calculation of Plume and Beam Temperatures

The 600WB190 beam directly above the fire is consider here. The radial distance of the beam from the fire, \( r \), is zero. Based on equations in Section 4.4.2, the temperatures of the flux onto the steel beam can be calculated.

An example of the calculation, based on the maximum flux (corresponding to the peak heat release rate of 17MW) is shown below:

\[
\begin{align*}
H_a &= \text{Height of lower flange of beam from the floor} = 2.85m - 0.6m = 2.25m \\
H_s &= \text{Height of fire source from the floor} = 0.3m \\
H &= H_a - H_s = 2.25m - 0.3m = 1.95m \\
Q_H &= \frac{Q}{1.11 \times 10^6 \times H^{2.5}} = 17 \times 10^6 / (1.11 \times 10^6 \times 1.95^{2.5}) = 2.88 \\
Q_D &= \frac{Q}{1.11 \times 10^6 \times D^{2.5}} = 17 \times 10^6 / (1.11 \times 10^6 \times 3.25^{2.5}) = 0.33 \\
z' &= 2AD \left(\frac{Q_D^{2/5}}{Q_H^{2/5}} - \frac{Q_D^{2/3}}{Q_H^{2/3}}\right) = 0.4 \\
L_H &= 2.9H(Q_H)^{0.33} - H = 2.9 \times 1.95 \times 2.88^{0.33} - 1.95 = 6.07m \\
y &= \frac{r + Hz'}{L_H + Hz'} = \frac{0 + 1.95 + 0.4}{6.07 + 1.95 + 0.4} = 0.28 \\
q^* &= 136000 \times 121000 \times 0.28 = 102,000W/m^2 \text{ (Capped at 100kW/m²)}
\end{align*}
\]

As done previously with a spreadsheet analysis, the flame temperature impinging on the steel can solved for the whole fire duration, as shown in Figure 7-2. The calculated temperatures at the different parts of the beam section are shown in Figure 7-3, as per the inputs in Section 5.5.3.1.
Figure 7-2: Calculated fire temperature above at the soffit of beam above the fire.

Figure 7-3: Calculated steel temperatures of beam located directly above the fire.

C.1.4 Structural Calculations

Flexural capacity check

The applied bending moment at midspan (8.5m from the end support of the beam) is:

\[ M_{\text{fire, midspan}} = \frac{Wx}{2}(1-x) = 647kNm \]
Where:

\[ W = \frac{\text{Applied Load}}{\text{m length}} = 17.9\text{kN/m} \]
\[ L = \text{full length of beam} = 17\text{m} \]
\[ x = \text{distance at point of interest from end support} = 8.5\text{m} \]

The variation of the flexural capacity of the composite beam as a function of temperature is summarised in Figure 7-4. The flexural capacity of the beam decreases and exceeds the applied moment at approximately 10 minutes; a plastic hinge forms at the beam midspan at this time.

Figure 7-4: Comparison of flexural capacity and applied bending moment for a utility vehicle at midspan.

This means that the beam would not be able to support the applied moments. Similarly, with Scenario 2, the ability of the adjacent beams to carry the remaining load by redistribution of the moments to the adjacent beams is checked, as shown schematically in Figure 7-5, whereby the adjacent beams are marked in blue.
Figure 7-5: Location of adjacent beams relative to main fire-exposed beam.

The temperatures of the adjacent beams is checked using the same procedure as outlined in Section 4.4, but using R (radius) as 3.8m.

Figure 7-6: Calculated steel temperatures of beam located radially 3.8m from fire.

The calculated steel temperatures of the top flange, web and bottom flange of the adjacent beams located 3.8m from the plume centreline are shown in Figure 7-6.
Figure 7-7: Comparison of flexural capacity and applied bending moment for each secondary beam located 3.8m from the fire location.

The calculated bending capacity of each of the adjacent beams is shown in Figure 7-7. The analysis shows that the capacity of the adjacent beams decrease, and the applied moments increase as the moments for the main heated beam are redistributed to the adjacent two beams. However the bending capacity of the adjacent beams is not exceeded.

Shear capacity check

The variation of the shear capacity of the composite beam as a function of temperature is summarised in Figure 7-8. The minimum shear capacity of the beam is 147kN, which is greater than the applied shear force, which is zero at midspan. Therefore, the beam is considered to have sufficient shear capacity at this location.
Figure 7-8: Comparison of shear capacity and applied shear force (Main beam directly above the fire).

C.1.5 Summary for Utility Vehicle Fire Scenario
The analysis presented here has shown that the unprotected steel secondary beams have sufficient flexural and shear capacity to resist the applied actions, when the structure is subjected to a utility vehicle fire.
C.2 Multi Vehicle Fire

C.2.1 Step 1: Design Fire

This section discusses the effect of a multi-car fire on a beam. It should be noted that as previously discussed in Section 4.2, a multi-car fire is considered likely in an unsprinklered carpark, but is considered here to show how it can be applied.

Figure 7-9 shows an example whereby three cars are involved, progressively burning from vehicle 1 and spreading sideways to the two adjacent cars (cars 2 and 3). It is assumed that fire spreads to cars 2 and 3 simultaneously. The impact of the three car fires on the single red beam is assessed. The heat release rate from the first car and the subsequent adjacent cars is shown in Figure 7-10. As discussed previously in Section 4.2, the fires are assumed to spread from the first car to the adjacent cars after 12 minutes.

![Diagram showing the progression of fire spread](image_url)

Figure 7-9: Schematic showing the progression of fire spread
C.2.2 Step 2: Check if the flame height will impinge on the underside of the beam

The flame height check is similar to that previously done in Section 5.5.2.

C.2.3 Step 3: Calculation of Plume and Beam Temperatures

The flux received by the beam from the first vehicle and the adjacent two vehicles is shown in Figure 7-11. Note that the flux received by Vehicles 2 and 3 are identical, hence they overlap on the graph. It shows that the fluxes received by the (red) beam from cars 2 and 3 are significantly lower than that of the car 1, which is located directly under the beam. The combined flux from all three vehicles and the resultant flame temperature received by the beam is shown in Figure 7-12. The calculated temperatures at various parts of the beam section are shown in Figure 7-13.
Figure 7-11: Received fluxes

Figure 7-12: Calculated total received flux fire temperature above at the soffit of beam above the fire.
Figure 7-13: Calculated steel temperatures of beam located directly above the fire.

C.2.4 Structural Calculations

Flexural capacity check

- The applied bending moment is calculated at 2.5m from the end support of the beam (directly above the fire), which is calculated as:

\[ M_{\text{fire, 2.5m}} = \frac{W \cdot x}{2} \cdot (L-x) = 325kNm \]

Where:

- \( W \) = Applied Load / m length = 17.9kN/m
- \( L \) = full length of beam = 17m
- \( x \) = distance at point of interest from end support = 2.5m

The variation of the flexural capacity of the composite beam as a function of temperature is summarised in Figure 7-14, which shows that the flexural capacity of the beam exceeds the applied moment at approximately 24 minutes; a plastic hinge forms at the beam at this time.
Figure 7-14: Comparison of flexural capacity and applied bending moment for a utility vehicle at midspan.

The ability of the adjacent beams to carry the remaining load by redistribution of the moments is checked as shown schematically in Figure 7-15, whereby the adjacent beams (#2 and #3) are marked in blue. These beams are also subjected to relatively high temperatures as cars 2 and 3 are located close to each of beams #2 and #3.

Figure 7-15: Location of adjacent beams relative to main fire-exposed beam.

The temperatures of the adjacent beams (#2 and #3) are checked using the same procedure as outlined in Section 4.3, but using the R (radius) from the plume to the beam as 1.25m.
Figure 7-16: Total received flux by each of beams #2 and #3

Figure 7-17: Calculated steel temperatures of beams #2 and #3

The calculated steel temperatures of the top flange, web and bottom flange of the adjacent beams located 1.25m from away from cars 2 and 3 are shown in Figure 7-17.
The calculated bending capacity of each of beams #2 and #3 is shown in Figure 7-18. The analysis shows that the capacity of beams #2 and #3 decrease, and the applied moments increase as the moments for the main heated beam are redistributed to the adjacent two beams. However the bending capacities of beams #2 and #3 are not exceeded and therefore the unprotected beams can resist the three-car fire in this location.

**Shear capacity check**

The variation of the shear capacity of the composite beam (#1) as a function of temperature is summarised in Figure 7-19. The minimum shear capacity of the beam is 123kN, which is greater than the applied shear force of 108kN. Therefore, the beam is considered to have sufficient shear capacity at this location.
C.2.5 Summary for Multi Vehicle Fire

The analysis presented here has shown that the unprotected steel secondary beams have sufficient flexural and shear capacity to resist the applied actions, when the structure is subjected to a multi vehicle fire.