THE APPLICATION OF ADVANCED FINITE ELEMENT ANALYSIS FOR STRUCTURAL FIRE DESIGN

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ABSTRACT

This paper presents a case study into the use of advanced finite element analysis as part of a performance based structural fire engineering design for the Justice and Emergency Services Precinct in Christchurch. The building is a multi-storey steel framed building with long span cellular beams supporting composite concrete floor slabs, and with unreinforced concrete filled steel hollow section columns. As part of the building design, the secondary cellular beams and the steel hollow sections are proposed to be unprotected. Advanced 3D finite element analyses were carried out to analyse the performance of the structure under exposure to realistic compartment fires. The analyses was able to consider the different realistic structural phenomena during the course of the fire including compressive restraint forces and contraction of the structural elements during the heating and cooling phases, and lateral buckling of the cellular beams in fire. This paper shows how the current knowledge of structures in fire and computational models have enabled engineers to positively influence the design of a building and to provide significant design information to structural engineers.

Introduction

The application of structural fire engineering analysis to building structures is one aspect of performance-based design that has been gaining momentum over the past two decades. With the advent of faster computing power and better numerical models, it has become feasible to undertake advanced analyses to understand the behaviour of how critical structures perform in real fires, and to inform the design of the building response to severe fire conditions.

Building Description

The Christchurch Justice and Emergency Services Precinct is a new five storey building, consisting of four moment-resisting steel framed structures that are built on a large podium structure (Figure 1). The Precinct consists of three parts - a justice building, an emergency services building and a parking building for operational vehicles. The precinct has stringent requirements for structural robustness for seismic and fire resistance, due to the Emergency Operations Centre located in the Emergency Services Building which will be a centre for emergency response and coordination in the event of a natural disaster.

The building structure consists of moment resisting frames with concrete filled steel CHS columns and I-Beams. The structural frame also features long span secondary cellular beams which support concrete-metal deck floors. The structural elements are required to achieve a 60 minute fire resistance rating. Conventionally, to achieve this required fire rating, all the structural columns and beams would need to be protected with a fire resisting protection material such as intumescent paint, fire resisting plasterboard or SFRM (Sprayed Fire-Resistive Material). As part of the design, an alternative fire protection strategy for the structure was proposed, featuring unprotected secondary steel beams and unprotected columns. An advanced analysis was undertaken to demonstrate the stability of the structure with the alternative fire protection strategy using the SAFIR non-linear finite element program.

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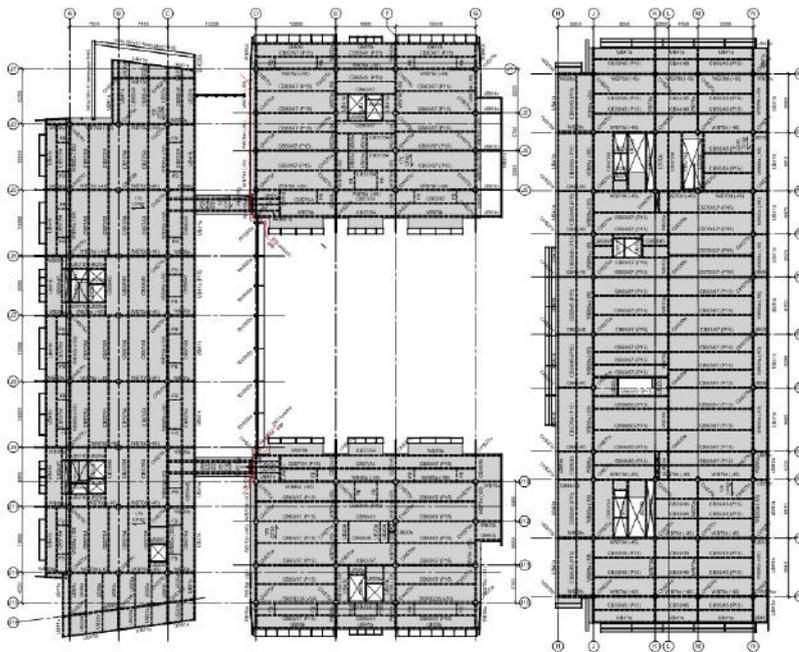[2] Holmes Fire, Level 1, 39 Market Place, Auckland, New Zealand, Martin.Feeney@HolmesFire.com

Figure 1: Beam layout on typical floor.

Steel Structures in Fires

Fire resistance in steel framed structures is conventionally achieved through the application of passive fire protection to the structural elements. The purpose of the insulating passive protection is to prevent high temperatures within the structural element. Up until the 1990s, it was generally perceived that a steel structure could suffer significant or catastrophic damage in a fire if the structure was not protected.

Part of this perception of structural behaviour originates from standard fire testing in furnaces for structural elements. These furnace tests, originally developed in the early 1900's, are useful for providing a benchmark of structural fire behaviour of individual elements under specific laboratory conditions. The standard fire curve was established based then based on what was perceived as a worst-case time-temperature relationship to be expected during a fire. The curve has remained essentially unchanged since and has been adopted by numerous countries around the world (ASCE 2009).

Such furnace tests are useful for benchmarking the performance of isolated structural elements of limited size. However, these tests do not provide an accurate representation of how structural elements within a real building respond under a realistic fire. For example, the test furnace structural supports do not represent those found in modern real buildings and the tested specimens are usually shorter than the long beam and column spans that those that are built in modern structures. The furnace tests also typically do not consider the cooling phase of a real fire which is important at these can produce significant contraction-induced forces when the elements cool down.

Since the 1990's, significant experimental and analytical research has been undertaken in Europe and Australia into the structural fire performance of whole steel buildings under real fire conditions (SCI 2000). This research showed that, if designed to mobilise multiple redundant load paths or utilising the inherent fire resistance of the entire structural frame, composite steel structural frames are not require to be fully fire protected and that some of the elements, such as the secondary beams, can be unprotected.

Advanced Numerical Methods

The research in the 1990's has also led to the development of advanced numerical models for analysing the highly complex nature of structural fire behaviour. These numerical models, typically based on the finite element method, have provided engineers and researchers with better insights into the physics of structural behaviour in fires.

These numerical models have progressively become more robust, and coupled with increasing computing power, have provided engineers with the ability to apply such analysis on commercial building and infrastructure projects. These programs include bespoke finite element programs such as SAFIR and Vulcan, which have been developed specifically for analysing structures in fire, or commercial general

purpose finite element programs such as ABAQUS or ANSYS, which have been modified for this purpose.

These programs can consider the highly non-linear nature of structures in fire, including large displacements and non-linear temperature dependent material properties, and can model the thermal expansion and contraction of the structural elements at elevated temperatures. They can incorporate advanced material properties which can consider the loss of strength and stiffness, damage, and recovering of strength and stiffness when the materials cool down.

Structural Fire Modelling of the Justice Precinct

A series of advanced analyses using non-linear finite element analysis was carried out to test the robustness and stability of the Justice Precinct structure under fire exposure and to investigate if the steel hollow section columns and the secondary steel beams could be unprotected. The analyses consisted of detailed heat transfer analysis of each structural component under exposure to a realistic post-flashover fire and 3D modelling of the heated elements within a structural frame.

The structural fire modelling was undertaken using the SAFIR finite element program. SAFIR (Franssen 2005, 2012) is a bespoke non-linear finite element program that has been developed at the University of Liege, Belgium, for thermal and structural analysis of concrete, timber, steel and composite structures in fire conditions. The program considers large displacement behaviour, non-linear temperature dependant material properties, and thermal expansion and contraction of materials at elevated temperatures. The program contains a range of structural finite elements for modelling typical civil engineering structures. It can model complex cross sections and the corresponding temperature distributions across the elements.

Definition of Design Fire

A realistic fully developed office fire was considered for the structural fire analysis for the Justice Precinct. The temperature-time fire curve was defined through the use of a Parametric Fire which considers the compartment characteristics such as the amount of ventilation, compartment linings and the expected fuel load. Through this, the fire growth, peak temperature and burning duration of a realistic fire is defined. The significance in using a realistic fire is that it provides a more realistic representation of the peak temperatures and duration of a fire in the building. In addition, the cooling of the elements during the decay phase of the fire would impose axial tension forces on the elements which would realistically represent the forces that could be present in a real fire.

One of the design fires used in the structural fire analysis is shown in Figure 2, which is based on an office occupancy. The design fuel load used in the analysis is 400MJ/m^2.
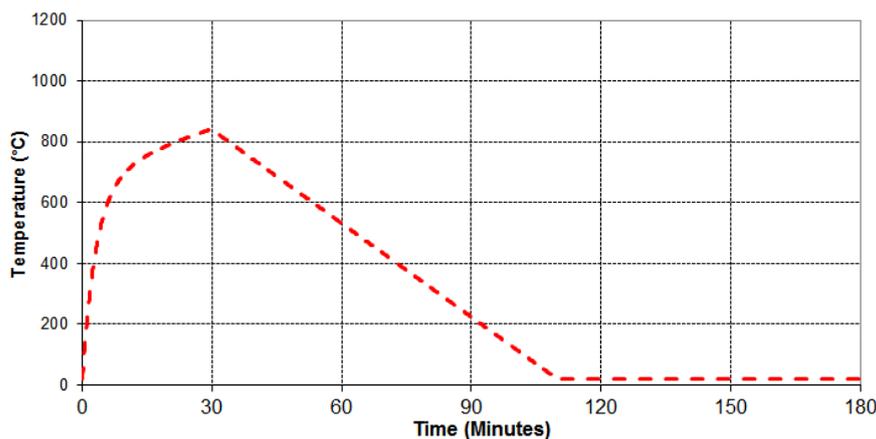

Figure 2: Design fire for office

Heat Transfer Analysis

Using the defined design fire, specific heat transfer analyses are undertaken for all the structural elements, as shown by an example in Figure 3. This includes the protected and unprotected beams, floor slab and composite columns. The heat transfer utilises the convective and conduction coefficients based on Eurocode 1 (BSI 2009) to calculate the detailed time-dependent temperature distribution across the structural sections. This allows the effective strength and stiffness degradation and thermal gradients across the structural section to be accurately defined. The ability to represent the temperature distributions accurately is also

critical in determining the behaviour of the structure. For instance, non-uniform temperature gradients in the elements can result in thermal bowing deflections and bending moments forming in the elements. If the temperatures are not accurately represented, it may result in an inaccurate response of the structure.
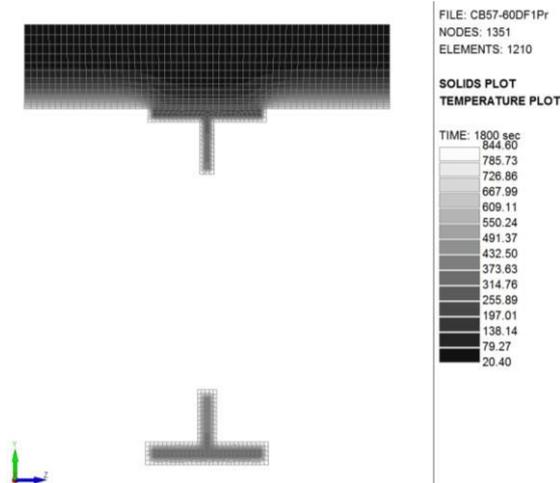

Figure 3: Typical heat transfer calculations undertaken in SAFIR for a composite cellular beam.
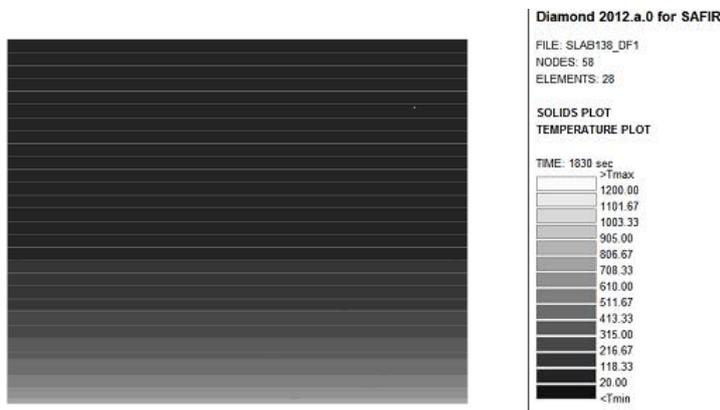

Figure 4: Temperature profile across equivalent 138mm flat slab at 30mins (Peak fire temperature for Office Fire).

The composite floor construction is modelled as a reinforced concrete flat slab with an effective thickness of 138mm, for simplification of the structural analysis. The flat slab is still able to represent the structural fire behaviour of the composite floor because under fire exposure, the metal deck will have negligible strength and stiffness and the composite floor will behave similarly to that of a reinforced concrete flat slab.

Figure 4 shows a finite element heat transfer model and the temperatures across the equivalent flat slab at 30 minutes into the design fire, which corresponds to the peak fire temperature. The cross section of the flat slab is represented as a rectangular slice across the slab thickness and the slab is modelled using rectangular solid finite elements, each approximately 5mm thick. The temperatures across the section of the slab at various stages of the fire are shown in Figure 5.

The thermal properties of the concrete are based on normal weight siliceous aggregate concrete based on EN1992-1-2 (BSI 2009), which is referenced in NZS3101 (2010) for assessing fire resistance. Spalling of the concrete, resulting in the loss of concrete cover, is not modelled. However, the provision of the metal deck, is expected to mitigate loss of cover due to spalling.
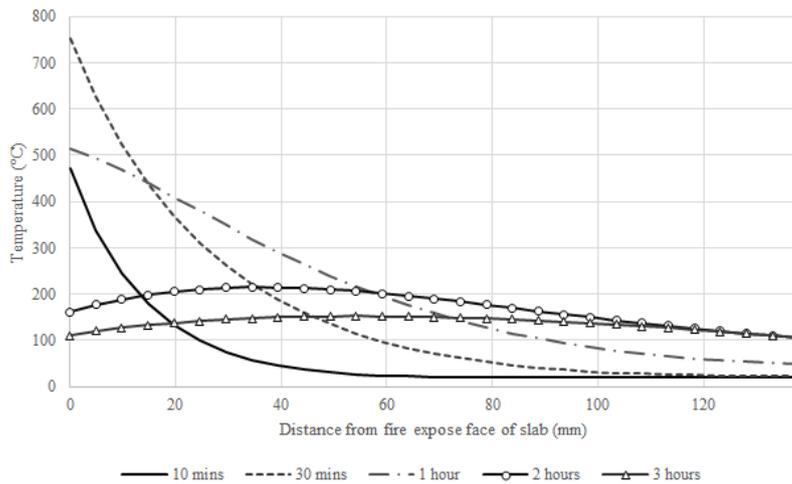
Figure 5: Temperature distribution across the flat slab under various stages of the design fire

Structural Analysis

The structural analysis of the frame is carried out based on the thermal analysis results incorporating the physical loads on the structure. The analysis of the structural frame calculates the structural response of the interconnected elements considering their loss of strength and stiffness, and thermal expansion and contraction of the elements as a result of high temperatures. It also includes the thermal strains, deflections and forces of the structure due to the high temperatures. Different sub-models have been analysed, instead of a large single structural model which covers all the conditions; this is to reduce the analysis time as this approach is more efficient for delivering results relevant to the design.

Figure 6 shows one of the structural subassemblies which have been modelled. This model assesses the structural fire performance of a typical structural configuration with the unprotected secondary beams. A subassembly of part of the building is modelled, instead of the entire building, due to the regularity of the structural frame. Therefore, the subassembly analysis is sufficient to model and represent the behaviour of the structure.
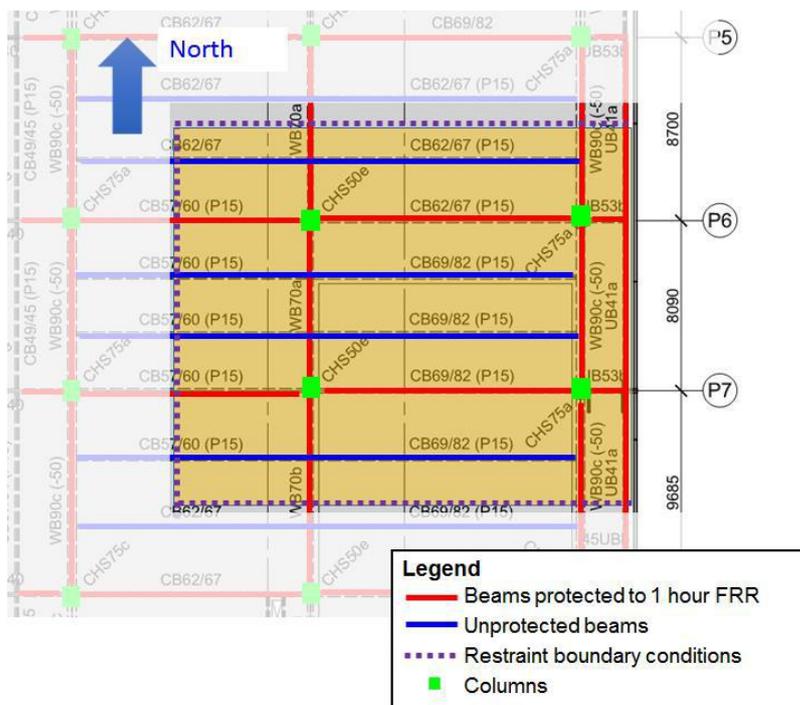

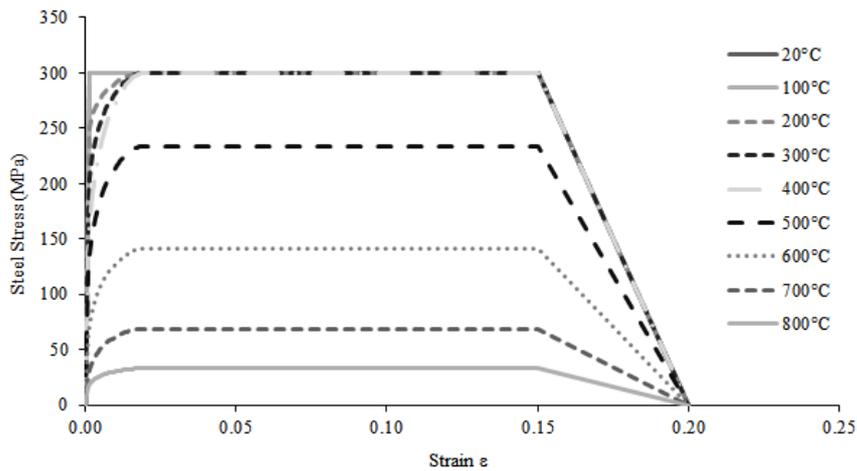Figure 6: Plan view showing the extent of a structural model

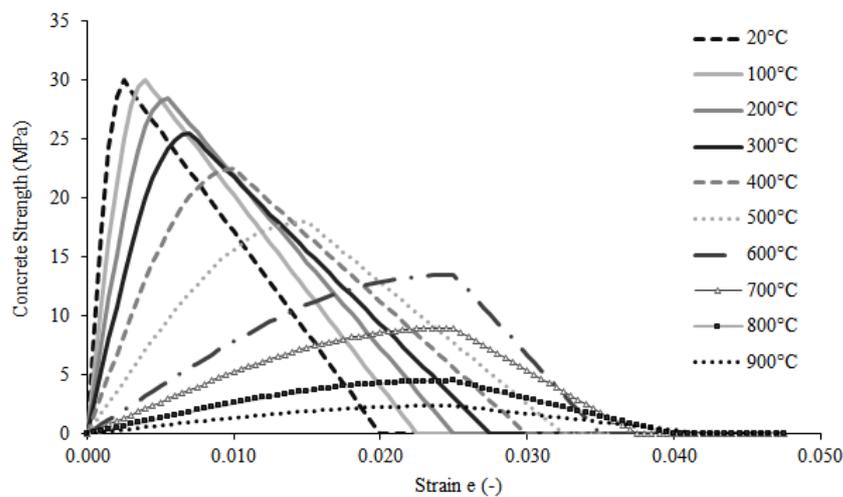Figure 7: Stress Strain Curve for Structural Steel at Elevated Temperatures

Figure 8: Stress Strain Curve for Concrete at Elevated Temperatures

Non-linear temperature dependant material properties are used in the structural analysis. Figure 7 and Figure 8 show the temperature dependent stress strain curves of the steel and concrete (in compression) that have been utilized in the calculations. These properties are based on the Eurocode 2 (BSI 2004) and Eurocode 3 (BSI 2005). The effect of cracking of concrete on the stiffness of the structural elements is considered in the structural fire analysis, modelled based on the smeared crack model.

To simulate the effects of lateral buckling of unprotected long span cellular beams in fire, a modified material property is used for the bottom tee of the cellular beams. The material has the same properties as structural steel (based on Eurocode 3) up to 500°C but loses its material properties between 500°C and 600°C. This modified material has been used by Gernay and Franssen (2010, 2013) in SAFIR to simulate unprotected long span cellular beams in fire and has shown good correlation when compared to full scale experimental fire tests.

The modelling of the lateral buckling of the bottom tee, and the associated loss of stiffness and strength contribution of the bottom tee is critical. If this phenomena is not taken into account, the strength and stiffness of the floor system would be overpredicted, resulting in unconservative results.
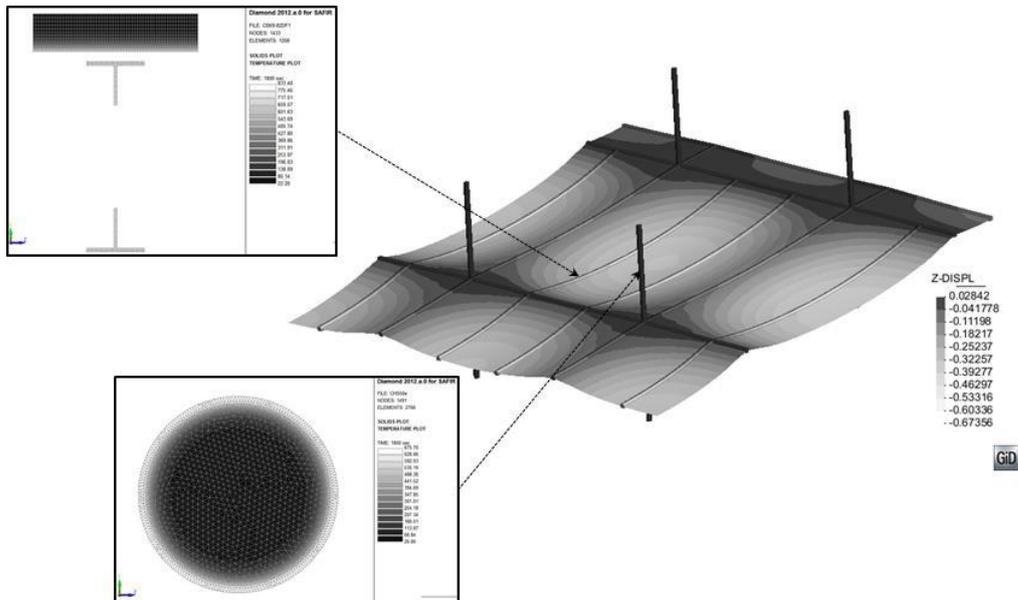
Figure 9: Deflection contours results of the 3D analysis of the structural steel frame with unprotected long span secondary beams during the peak fire temperature (30 minutes).
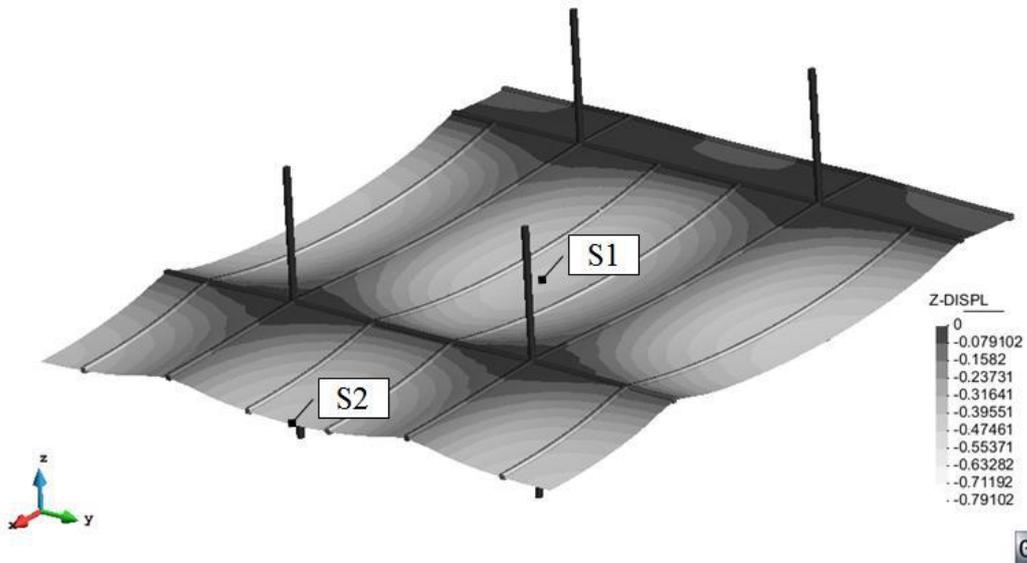

Figure 10: Deflection contours results of the 3D analysis of the structural steel frame with unprotected long span secondary beams at the end of the analysis (3 hours).
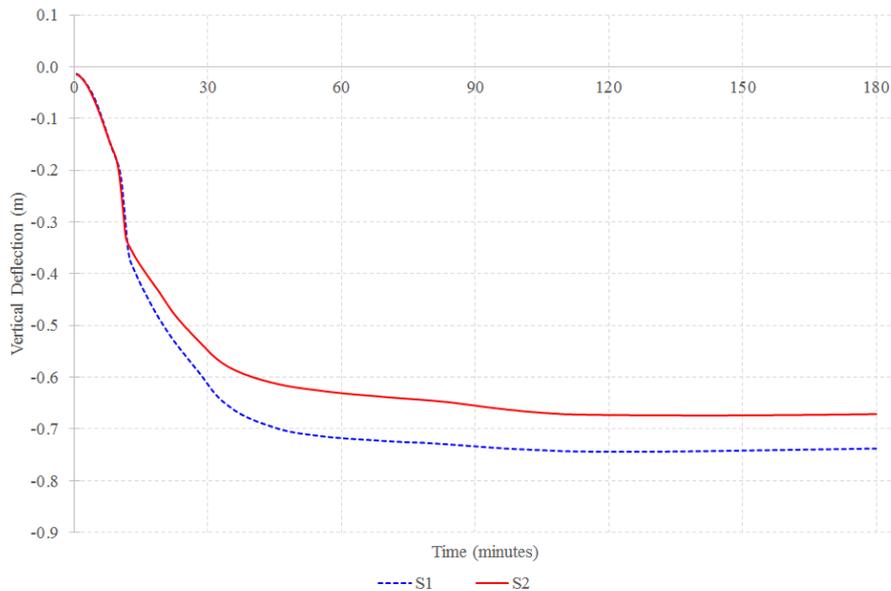
Figure 11 Variation of midspan vertical deflections with time at midspan locations S1 and S2 (Refer to Figure 12 to locations)

Figure 9 and Figure 10 shows the vertical deflections of the structure at the peak of the fire temperature (30 minutes) and at the end of the cooling phase (3 hours). At the end of the fire (3 hours), the fire has decayed and the temperatures of the structural elements have decreased and allowed the unprotected steel beams to fully regain its strength and stiffness. The beams do not recover to their initial undeformed state after the fire, when the structure has cooled, due to permanent plastic deformations in the unprotected beams and the slab (See Figure 11). Figure 11 shows the vertical deflections at various areas of the floor slab, as indicated in Figure 10. It shows large downward deflections during the first 15 minutes of the fire, due to the loss of strength and stiffness of the unprotected secondary beams. Connector elements were specified at the ends of the unprotected secondary beams with temperature dependent axial capacities. The connector elements were modelled with temperature dependent axial capacities and monitored to check if they could fail by exceeding specified failure limits corresponding to bolt failure.

The ability of the floor construction with the unprotected secondary beams to resist the applied loads and the design fire is due to tensile membrane action forming in the slab. This behaviour has been demonstrated by numerous experimental fire tests (Nadjai et al 2011, SCI 2000) and by numerical modelling (Huang et al 1999, Lim et al 2004, Gernay et al 2010). Figure 12 show the development of membrane forces in the slab of the bays which form the load resisting mechanism whereby the tension forces (light coloured lines) in the middle of the bays are resisted by the compression forces (dark coloured lines forming around the perimeter of each bay). Using 3D modelling of a structural frame, the inherent fire resistance provided through full frame behaviour can be quantified. It shows that the structure has sufficient resilience to resist the loads in the fire limit state.
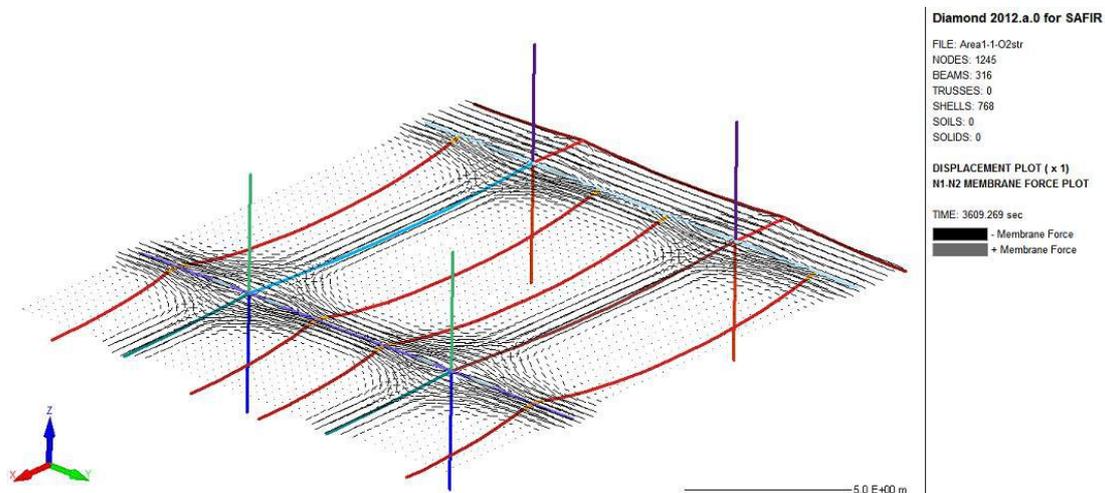

Figure 12: Membrane forces forming within the slab during the peak of the fire (Dark lines refer to compression membrane forces, light lines are tensile membrane forces).

The outcome of such analysis has provided insights into the performance of the structure in fire conditions, which would not be possible with standard prescriptive designs. For instance, the analysis has been able to estimate the structural deflections and temperature-induced compressive restraint and tensile forces throughout the fire exposure. The ability to detect such forces allows engineers to be able to design for and mitigate failure, which is particularly critical in a building of high importance. This provides stakeholders (approving authorities, structural engineers, insurers) with a better level of understanding of how the structure would respond during a fire.

Conclusions

The ability to quantify the fire behaviour of structures has improved significantly over the last two decades with the development of advanced numerical models. As shown in the case study of the Christchurch Justice and Emergency Services Precinct, the outcomes of applying advanced analyses on such projects have resulted in designs which demonstrate their robustness as a whole structural system instead of relying on the fire resistance of individually protected elements.

The 3D modelling enabled different types of load resisting mechanisms to be considered, such as tensile membrane action, which would typically not be considered with simpler calculation methods. This allowed the structural design to be safely optimised, whilst avoiding over-design. The analysis was also able to identify issues, which lead to a more robust design, compared with a piecemeal approach based on simple assumptions of single element behaviour from simple furnace tests.

It has also been able to demonstrate that the secondary steel beams and steel columns do not require passive fire protection, which provides savings in fire protection and building materials, improved building aesthetics and greater architectural freedom.

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