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 a multi-storey steel framed building in New Zealand. The building is a multi-storey steel framed building with long span cellular beams supporting composite concrete floor slabs. As part of the building design, the secondary steel beams and composite columns of the structure are proposed to be unprotected. A series of advanced finite element analysis using the SAFIR finite element programis carried out to test the robustness of structure without passive fire protection for the secondary beams and composite columns. The numerical modelling features thermal modelling of the structural elements and 3D structural modelling of the heated elements to test the behaviour of the long span beams under realistic compartment fires. The analysis utilizes non-linear temperature dependent materials to consider realistic behaviour of the structural response under a fully developed compartment fire, including the rapid thermal loading during flashover and the cooling phase of the fire. The analysis is able to consider the different realistic structural responses during the course of the fire including contraction during the cooling phase and lateral buckling of the cellular beams in fire. This paper shows how such an analysis can be applied by consulting engineers on a realistic building design to demonstrate a robust design for a steel framed building with unprotected structural elements whilst providing savings with reduced amount of fire protection to the structure.

KEYWORDS

Concrete filled steel tubes, cellular beams, composite construction, finite element analysis, fire, structural fire engineering, steel.

INTRODUCTION

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 from forming within the structural element which will compromise its load bearing capacity. Until the 1990s, it was generally perceived that unless fully protected, an entire steel structure could suffer significant or catastrophic damage in a fire.

Part of this perception originates from standard fire testing in furnaces for structural elements which showed the poor behaviour of unprotected steel elements tested in isolation. These furnace tests, originally developed in the early 1900's, are useful for providing a benchmark of structural fire behaviour of individual elements under laboratory conditions. The standard fire was established based on what the American Society of Testing Materials described 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 only 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 beam and column spans that are built in modern structures. The furnace tests also do not consider the cooling phase of real fires which is important at these can result in high contraction forces when the elements, particularly beams, cool down.

STEEL STRUCTURES IN FIRES

Since the 1990's, significant experimental and analytical research has been undertaken in Europe (SCI 2000, Nadjai et al 2011) and Australia (Proe et al, 1994) into the structural fire performance of whole steel buildings under real fire conditions. This research showed that, if designed to mobilise multiple redundant load paths and other forms of load resisting mechanism, composite steel structural frames do not need 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 Europe in the 1990's has also led to the development of advanced numerical models for analysing the highly complex nature of structural fire behaviour (Huang et al 1996, Gillie et al 2001, Izzuddin et al 2002). These numerical models, based on the finite element method, have provided engineers and researchers with better insights into the physics of structural behaviour in fires.

These programs consist of either bespoke finite element programs such as SAFIR (Franssen 2005) or Vulcan (Huang et al 2004), 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 behaviour of structures in fire, including large displacements and non-linear temperature dependent material properties, as well as the thermal expansion and contraction of the structural elements at elevated temperatures. The programs can consider the loss of strength and stiffness in materials, damage, and recovery of strength and stiffness when the materials cool down.

Such programs have progressively become numerically more stable and robust, and coupled with increasing computing power, have provided consulting engineers with the ability to apply such analysis on commercial building and infrastructure projects.

Using advanced analyses, the inherent fire resistance provided by the building as a whole can be analysed, considering the actual structural design of the building, and the fire and structural loading conditions. Using such tools, consulting engineers can test the robustness of the structure in fire conditions. The outcomes of the analysis can give insights into the performance of a structure in fire conditions, which would normally not be possible with standard prescriptive designs or with simpler analysis methods. For example, such programs can include the effect of significant compression restraint forces which form during the heating phase as well as tension forces during the cooling phase. These forces can cause premature failure of the structural elements and the ability to detect such forces allows engineers to be able to improve their designs to mitigate failure. Such insights provide project stakeholders (Certifying authorities, architects, insurers) with better understanding of the true structural fire performance rather than a piecemeal approach to achieving fire resistance.

The application of such analyses also has other significant benefits for optimising or eliminating fire protection in steel framed buildings. This provides better building aesthetics by having exposed steel elements. This also reduces waste of materials and time through the reduction of unnecessary fire protection and also results in significantly less capital and life cycle costs, and shorter construction period. Advanced analysis has also been applied to concrete structures to reduce and optimise the thicknesses of reinforced concrete slabs, to quantify the residual strength of fire damaged structures and to design the structural elements for resisting high challenge fires such as hydrocarbon fires.

APPLICATION OF STRUCTURAL FIRE ENGINEERING

An advanced analysis was undertaken for the new Christchurch Justice and Emergency Services Precinct in New Zealand. The Precinct consists of three buildings, a Justice building, an Emergency Services building and a parking building for operational vehicles. The Justice and Emergency Services Building is a five storey building, consisting of four moment-resisting steel framed structures that are built on a large podium structure (Figure 1). The moment resisting frames consist of concrete filled steel CHS columns and I-Beams. The structural frame also features long span cellular beams and I-beams arranged in a regular grid, which in turn support concrete-metal deck floors.

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 structural elements are required to achieve a 60 minute fire resistance rating. Conventionally, all the columns and beams would need to be protected with a fire resisting protection material such as intumescent paint or SFRM (Sprayed Fire-Resistive Material) to achieve this fire rating. However, an alternative design was proposed where the secondary steel beams and the composite columns were not protected with passive fire protection.

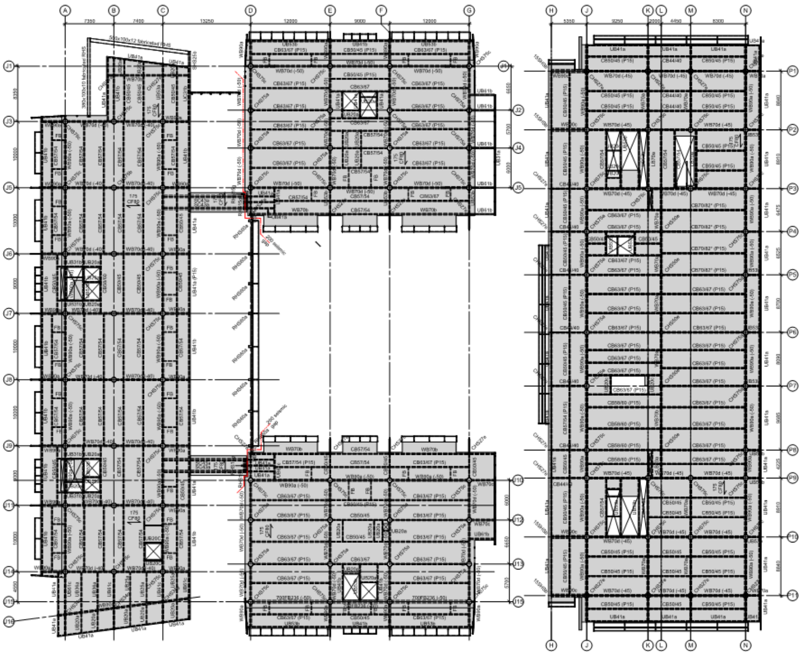

Figure 1: Beam layout on typical floor.

METHODOLOGY

A series of analyses using non-linear finite element analysis was carried out to test the robustness and stability of the structure in fire conditions and to quantify if the steel hollow section columns and the secondary cellular beams could be unprotected. The analysis consisted of detailed heat transfer analysis of the structural components under exposure of a realistic fire and 3D structural modelling of the frame to determine the structural response. The numerical analysis 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 the thermal and structural analysis of concrete, 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 incorporates pre-defined non-linear material properties, based on the Eurocodes, but also allows user-defined material properties for the analyses. The program contains a range of structural finite elements, such as beam elements and shell elements, for modelling civil engineering problems. The structural finite elements can consider non-linear temperature distributions across the cross section of the structural elements (e.g.: across beams, columns and slabs).

DESIGN FIRE

A realistic fully developed office fire was considered for the structural fire analysis. The temperature-time fire curve was defined based on the Eurocode 1 Parametric Fire (BSI 2009) definitions. The Parametric Fire represents a fully developed fire, whereby the rate of the fire growth, peak temperature and burning duration are considered through the amount of ventilation, compartment linings and the expected fuel load. 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 beams and the connections; this is a key structural phenomena which needs to be considered to check the adequacy of the beam connections. The design fire is shown in Figure 2 which is based on an office occupancy.

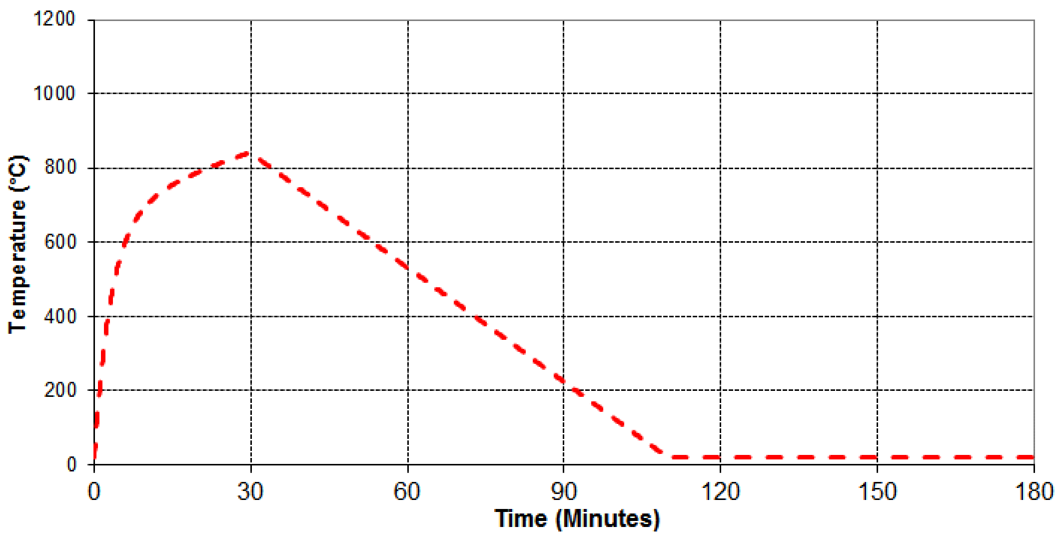

Figure 2: Design fire for office

Heat Transfer Analysis

Using the defined design fire, specific heat transfer analysis is undertaken for all the structural elements (Figure 3), including the protected and unprotected beams, floor slab and composite columns. The cross section of each heated structural element is discretised into finite elements, as shown in Figure 3. Each finite element within the cross section is assigned with its specific material property.

The heat transfer utilises the convective and conduction coefficients based on Eurocode 1 (BSI 2009). The heat transfer analysis calculates the temperature distribution across and thermal gradients across the structural section as a function of time. Non-uniform temperature gradients in the elements can result in deflections and bending moments forming in the beams and slabs, so an accurate thermal analysis is needed to be able to accurately determine the structural fire behaviour. The results of the heat transfer analysis are used to determine the strength and stiffness degradation in the elements during the implementation of the structural analysis.

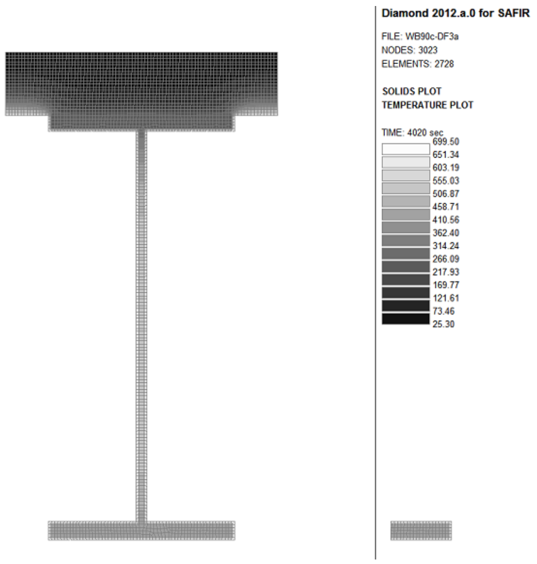

Figure 3: Typical heat transfer calculations undertaken in SAFIR for a composite beam.

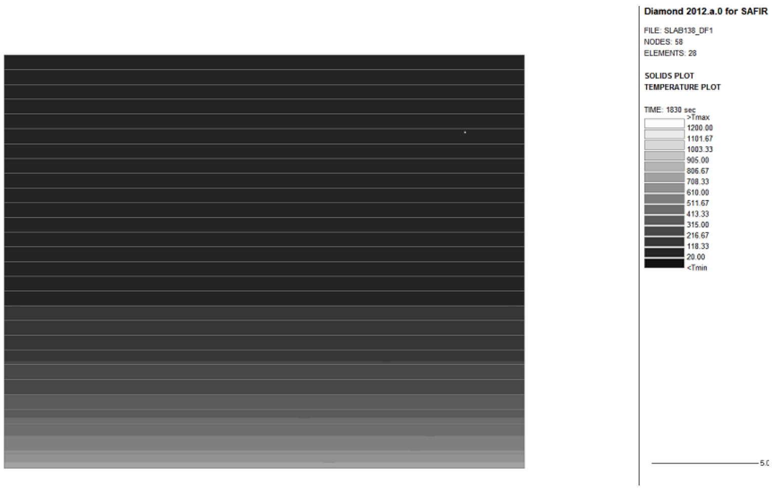

Figure 4: Temperature profile across equivalent 138mm flat slab at 30mins (Peak fire temperature).

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 effective thickness of the slab is determined in accordance with Eurocode 4 Part 1-2:2005 (BSI 2005). 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 temperatures across the section of the slab at various stages of the fire are shown in Figure 5. The cross section of the flat slab is represented as a rectangular slice across the slab thickness. The slab is modelled using rectangular solid finite elements. The thickness of each of the solid elements is approximately 5mm thick.

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 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.

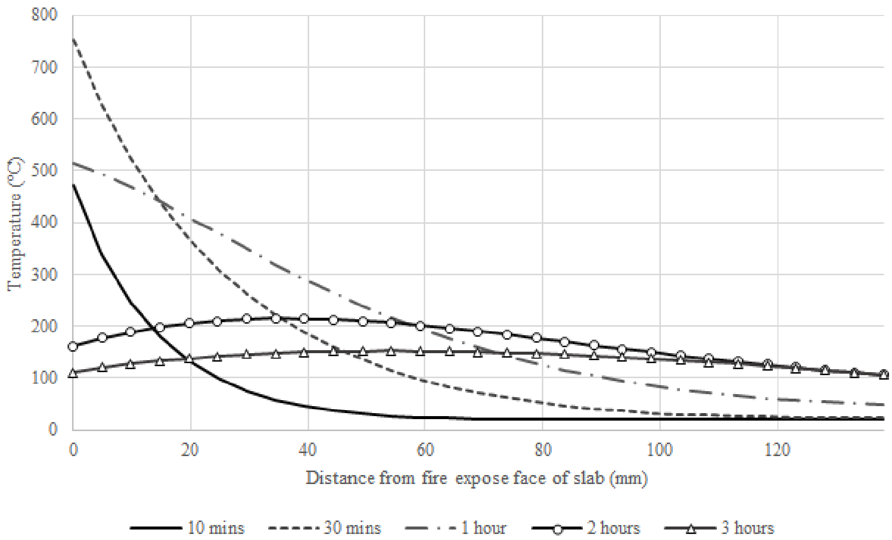

Figure 5: Temperature distribution across the flat slab under various stages of the design fire

STRUCTURAL ANALYSIS

The structural analysis of the frame is carried out based on the thermal analysis results of the members, incorporating the physical loads on the structure. The structural analysis calculates the structural response, including the thermal expansion strains, deflections and forces of the heated structural elements based on the outputs of the thermal analyses. It considers the loss of strength and stiffness, and thermal expansion and contraction of the elements as a result of high temperatures. For this project, different sub-models were 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 the plan view of one of the structural subassemblies which were modelled using the SAFIR structural fire analysis. The purpose of this model is to assess the structural fire performance of a typical structural configuration with the unprotected secondary beams and composite columns.

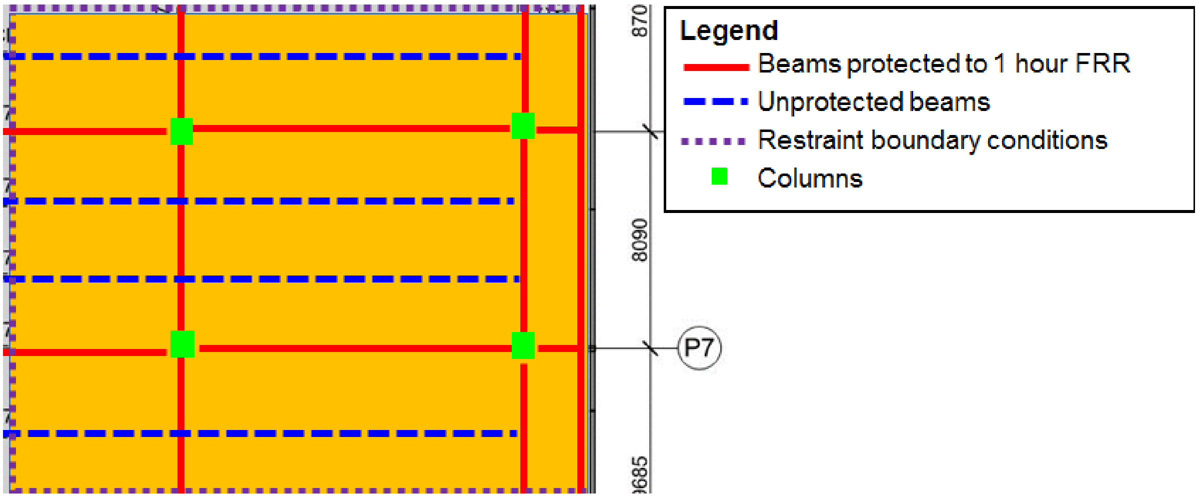

Figure 6: Plan view showing the extent of a structural model

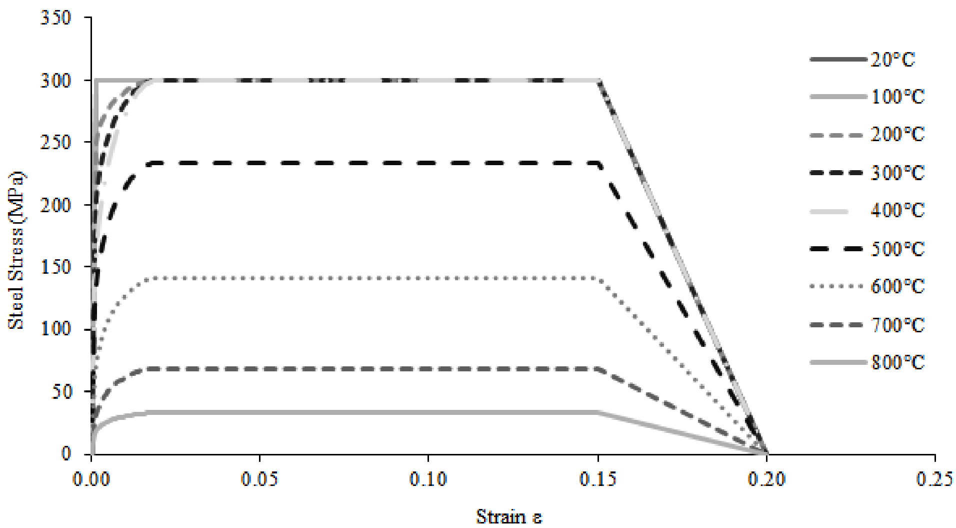

Figure 7: Stress Strain Curve for Structural Steel at Elevated Temperatures

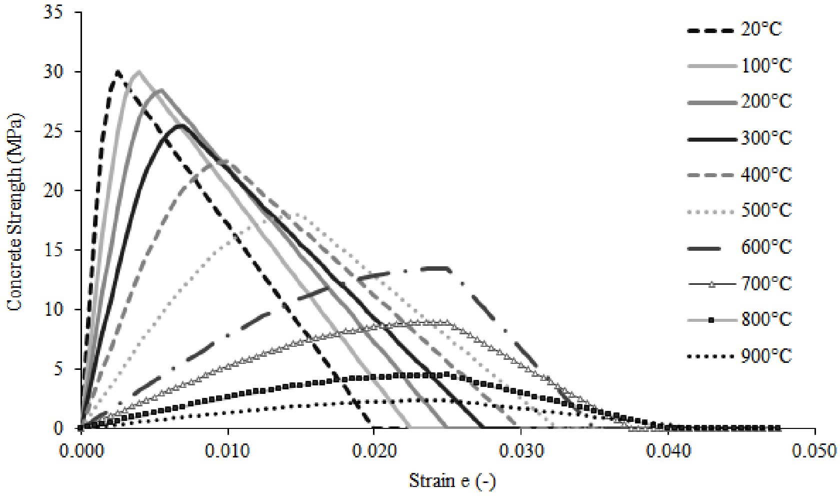

Figure 8: Stress Strain Curve for Concrete at Elevated Temperatures

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

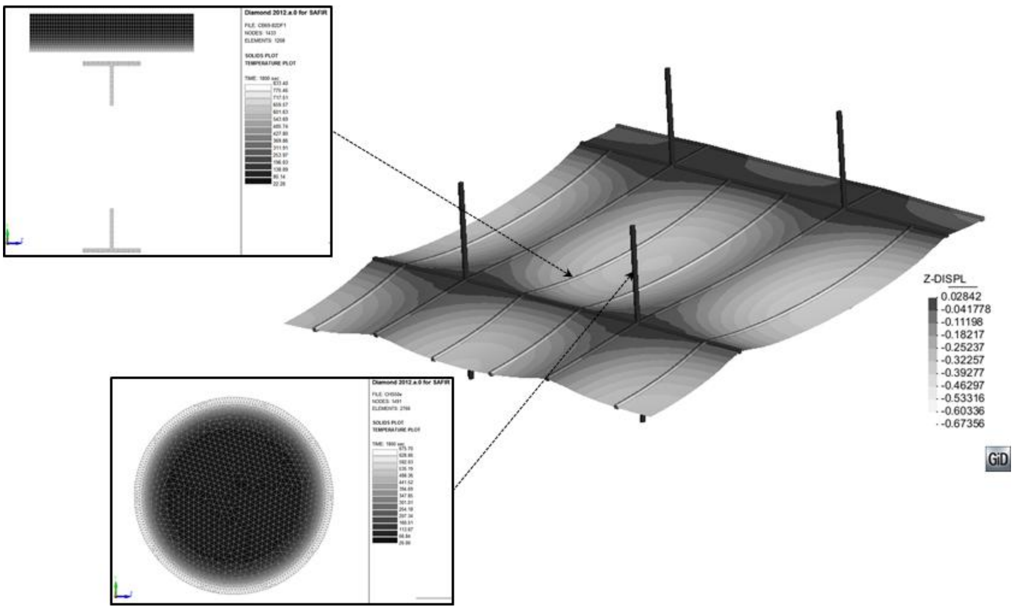

Figure 9: Deflection contour results of the 3D analysis of the structural steel frame with unprotected long span secondary cellular beams and composite columns shown at the peak fire temperature (30 minutes).

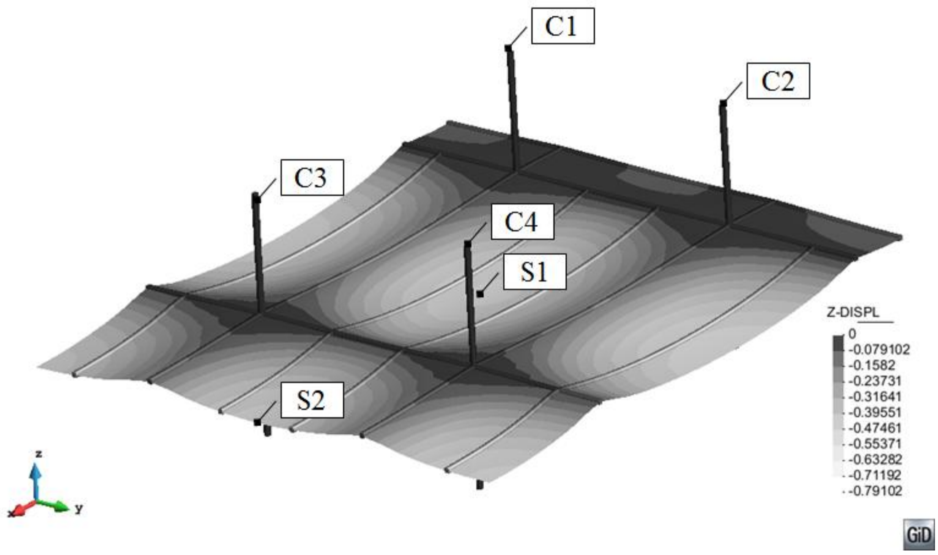

Figure 10: Deflection contour results of the 3D analysis of the structural steel frame at the end of the analysis (3 hours).

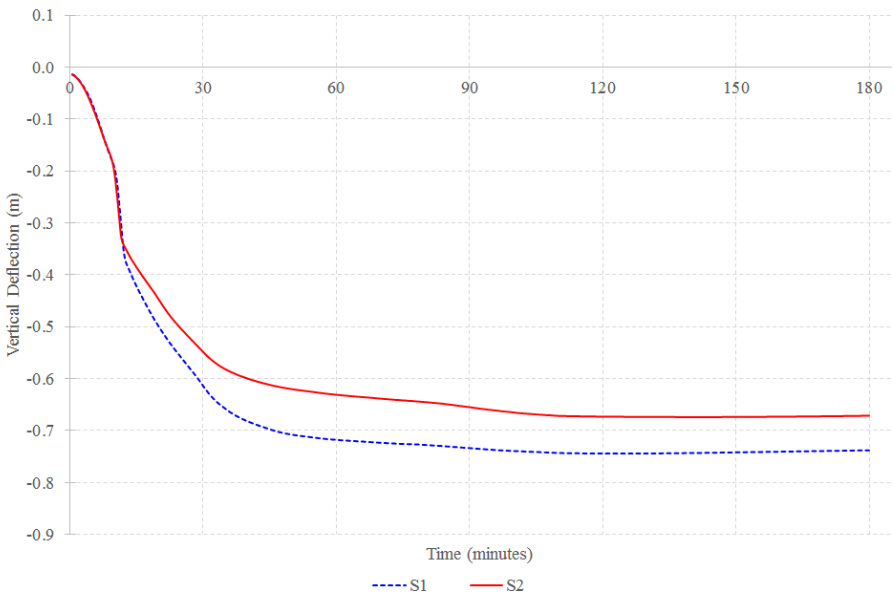

Figure 11 Variation of midspan vertical deflections with time at midspan locations S1 and S2 (Refer to Figure 10 for locations)

Figure 9 and Figure 10 shows the vertical deflections of the structure at the peak of the fire temperature (30 minutes) and at 3 hours, when the simulation was terminated. The analysis was performed for up to 3 hours, even though the fire had fully decayed after 2 hours, to capture the cooling of the structural elements. At the end of the simulation, the beams do not recover to their initial undeformed state due to permanent plastic deformations in the unprotected beams and the slab (See Figure 11). However, the analysis results show that the structure remained stable despite the large deflections in the floor structure. The results indicate that there was no runaway failure of any part of the structure. The stability of the structure is also depicted by the relatively small axial movements of the columns, as shown in Figure 12. The difference in the amount of axial movement of the columns is attributed the amount of axial load applied to the columns.

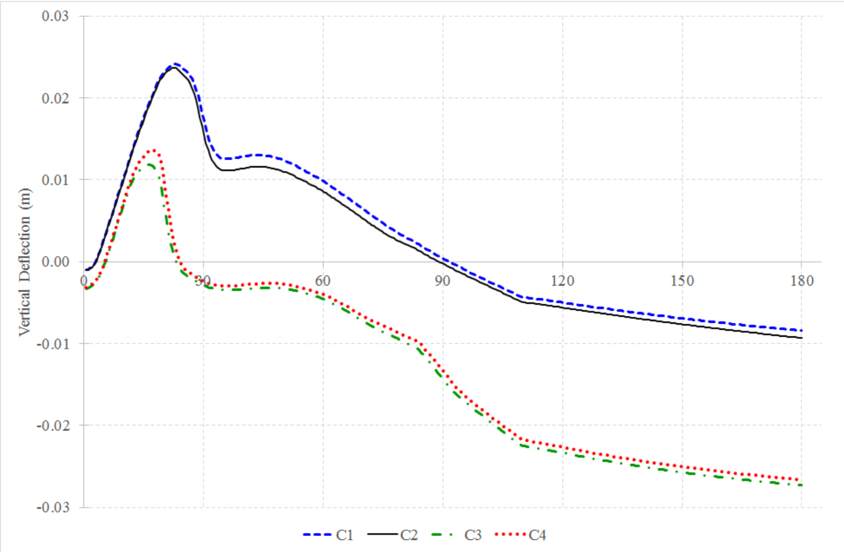

Figure 12Variation of vertical deflections with time at the top of columns (Refer to Figure 10 for locations)

Connector elements were also modelled at the ends of the unprotected secondary beams. The connector elements were modelled with temperature dependent axial capacities with a failure limit of 20mm, corresponding to the deformation limit of the bolts in the connections. The analyses showed that the axial capacities of the connections was reached, however, the axial deformations did not exceed the connection deformation limits.

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 experimentally in fire tests (SCI 2000, Nadjai et al 2011). The loads on the floor are resisted by the slab reinforcing in the central (tension) region, which is balanced by the compression ring around the edges of the rigid structural bay. This mechanism allows the floor structure to remain stable even when the unprotected secondary steel beams within the bays have lost significant bending strength and stiffness. Figure 13 shows the distribution of membrane forces in the slab, forming tensile membrane action. 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. This is pattern of forces is consistent with the findings of numerical analyses by numerous researchers (Huang et al 1999, Lim et al 2004).

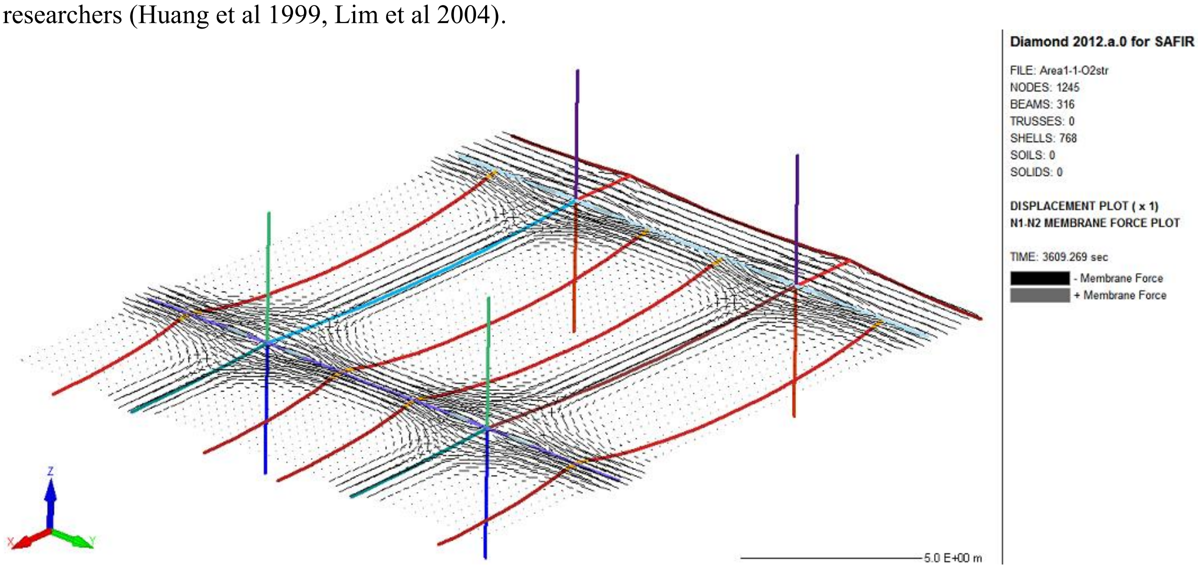

Figure 13: Membrane forces forming within the slab during the peak of the fire (Dark lines refer to compression membrane forces, light lines are tensile membrane forces).

This analysis demonstrated how a performance based analysis can be applied on a commercial project to provide a robust design by identifying structural weaknesses, but also enabled the structural design to be safely optimised, whilst avoiding over-design. The 3D modelling enabled different load resisting mechanisms to be included in the analysis, such as tensile membrane action, which would typically not be considered with simpler calculation methods. The application of such an analysis provided cost savings to the project through the reduction of passive fire protection to the secondary beams and columns.

CONCLUSIONS

The ability to quantify the fire behaviour of structures has improved significantly over the two decades with the development of advanced numerical models. Such numerical models can now be utilised by engineers on projects to demonstrate a robust and optimised design. As shown in the case study in this article, the outcomes of applying advanced analyses on commercial projects have resulted in designs which achieve robustness as a whole structural system, and providing savings in fire protection and building materials and improved building aesthetics.

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