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# Overview of the Benefits of Structural Fire Engineering

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**Abstract**

The field of structural fire engineering has evolved within the construction industry, driven largely by the acceptance of performance-based or goal-based design. This evolution has brought two disciplines very close together - that of structural engineering and fire engineering. This paper presents an overview of structural systems that are frequently adopted in tall building design; typical beams and columns, concrete filled steel tube columns and long span beams with web openings. It is shown that these structural members require a structural analysis in relation to their temperature evolution and failure modes to determine adequate thermal protection for a given fire resistance period. When this is accounted for, a more explicit understanding of the behaviour of the structure and significant cost savings can be achieved. This paper demonstrates the importance of structural fire assessments in the context of tall building design. It is shown that structural engineers are more than capable of assessing structural capacity in the event of fire using published methodologies. Rather than assumed performance, this approach can result in a safe and quantified design in the event of a fire.

**Keywords:** Structural fire engineering, Optimisation, Passive fire protection, Intumescent coatings

## 1. Introduction

Historically, the issue of structural fire resistance in building design has been assumed to be within the remit of the architect. In many instances this aspect of design involves no more than prescribing a required period of fire resistance for structural elements from a lookup table in a code or guidance document. Such lookup tables relate to the building occupancy (to equate the risk of fire and potential fire loads), the building height above ground and its depth below ground (in relation to evacuation and fire fighting activities) and the presence of a suppression system (which may act to control the development of a fire). The required fire resistance is therefore not a function of the characteristics of the structure. In the case of steel, these time periods of fire resistance are then typically incorporated into the structural steelwork specifications or may be indicated on general arrangement drawings. The whole process can be very quick and typically no further thought is given to it from a design perspective. Specifications are then sent to passive fire protection suppliers as part of a tender process and the associated costs are factored into the overall building price.

Over recent years however, there has been a change in attitude towards structural fire resistance. With the emergence of performance based design, a new breed of structural engineers has embraced the concept of robust design at elevated temperatures. This approach can result in significant economic savings, but more importantly it provides a basis for an informed, robust and safe structural design.

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### 1.1. Drivers for structural fire engineering

In the context of fire safety, the primary issue in terms of design is the life safety of occupants in a building. To a lesser extent, the role of property or asset protection can be important in the event of a fire. The issue of cost however, cannot be neglected and one of the main reasons that a structural fire engineering assessment is undertaken is to reduce overall project costs while maintaining adequate life safety performance. It is not an easy task to identify the costs associated with a particular structure, particularly at early stages of design, given that the structural design normally does not include thermal considerations. In many cases, it is not until latter stages of design that fire protection costs may be priced - at that point, an exercise in value engineering may take place that may include structural fire engineering.

Passive fire protection to structural steelwork is not necessarily a cheap solution. It is important to have a high-level understanding of associated costs to appreciate the benefit that a structural fire engineering assessment can have if it is considered early in design. Fig. 1 provides a breakdown of costs of components in a shell and core design for a typical 14-storey office building in central London (Building Magazine, 2011). This shows that 12% of the shell and core costs are associated with the structural frame of the building, which incorporates passive

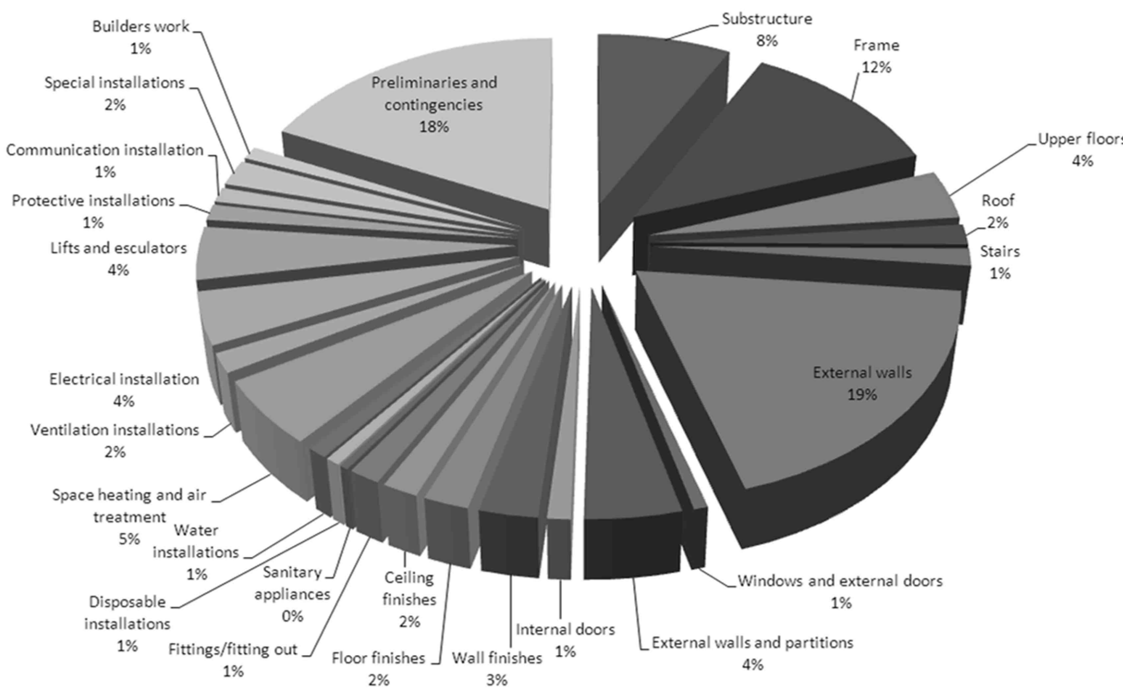


**Figure 1.** Example relative cost breakdown of components of a shell and core construction for a 14-storey office building in central London (Building Magazine, 2011). Fire protection costs are associated with the structural frame.

fire protection. Fig. 2 then breaks down the structural frame costs into its relevant components which indicates that fire protection accounts for approximately 20% of the structural frame cost, or rather approximately 2% of the total shell and core costs. It is quickly evident that for a tall building with a large budget, fire protection can form a significant cost. Therefore, it is not surprising that designers are looking to reduce costs through value engineering accounting for structural fire assessments. Such approaches have been published by leading consultants (Lamont et al., 2006).

The cost breakdown in Figs. 1 and 2 are provided for illustrative purposes only. A full comprehensive cost evaluation would need to take into account the total volume of fire protection together with associated application rates. This level of detail is likely to require close coordination

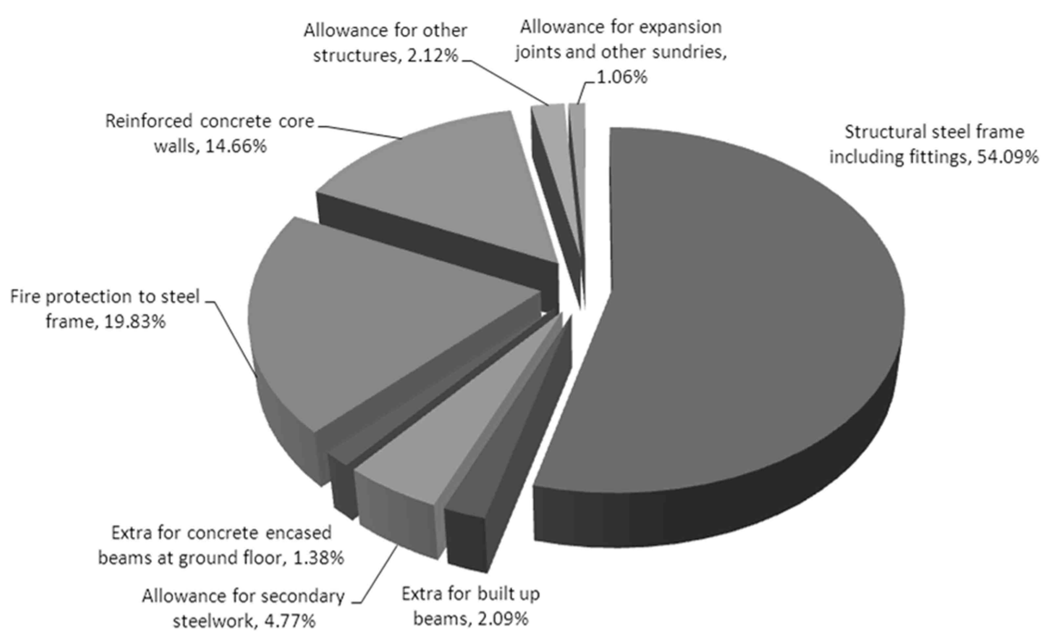


**Figure 2.** Example relative cost breakdown of components of the structural frame cost for a 14-storey office building in central London (Building Magazine, 2011) showing the proportion of cost associated with fire protection.

by the structural engineer, a passive fire protection manufacturer, the steelwork fabricator, an applicator and the general contractor.

### 1.2. Optimisation

Structural engineers often design for the most efficient use of structural steelwork for the variety of structural load cases or load combinations that they are considering. Efficient in this context is taken as being the lightest steel member, while load combinations are likely to incorporate dead, super dead, imposed, wind, snow and potentially seismic loads (EN, 2002). Most structural engineers have a good grasp of the price of steelwork and can therefore come up with indicative steelwork costs for their designs. However, few structural engineers at present have sufficient understanding of passive fire protection and their associated costs to appreciate what impact fire protection has on the cost of their designs and the potential for optimisation.

Fig. 3 illustrates a high-level cost comparison for a single steel member in terms of steel cost and fire protection cost. If the weight of a member is increased, for example an increase in the serial size with respect to its weight or an increase in the web or flange thickness, then it is obvious that the cost of the steel will increase. However, the increase in steel weight can generally have a large impact on the amount of fire protection required. A heavier (thicker) steel section will heat more slowly than a light (thin) steel section. As such, it will have an increased inherent fire resistance and therefore require a comparatively thinner thickness of applied fire protection to achieve the same level of required fire resistance. From Fig. 3 it is intuitive that there is likely to be an optimum weight that results in the cheapest design by factoring steel and fire protection costs together.

When considering fire protection costs it is important that the overall volume of material for a project is not priced in isolation. The application costs of various fire protection options are important too.

A structure that has been subject to a structural fire assessment should ideally take a full cost benefit analysis into account to arrive at the optimum design. This should account for the combined cost of steel, fire protection and application costs as a minimum.

## 2. The Structural Engineer and the Fire Engineer

Structural fire engineering is often seen as a specialist sub-division of structural engineering or fire engineering. Nevertheless, in many instances the structural fire engineer may typically be the fire engineer on a project. In addition to defining a building's fire safety strategy, the fire engineer may additionally consider the structural fire resistance and propose a performance-based design to reduce the fire resistance period (e.g. 120 minutes to 90 minutes) or the removal of specific portions of fire protection. While the boundaries between the professions are not well defined, the skills and tasks are.

It is the role of the fire engineer to be able to assess the severity of a potential fire affecting a structure. This can be done based on many different calculation methods that take into account the potential fuel, the degree of ventilation, the compartment geometry and the possible presence of a suppression system. The most commonly used methods will treat the fire as a homogeneously heated compartment (EN, 2002) but alternate methods (BS, 2001) may include flame height development from a localised fire or flame projection from an opening. In all cases, the maximum temperature or temperature evolution of steelwork can be assessed through heat transfer calculations (EN, 2005). All these calculation methods bring strong assumptions that can be relaxed by using more detailed methods such as computational fire models. The technical literature presents many of these methods which are reviewed by (Torero, 2013).

The structural engineer should ideally determine the limiting steel temperature of each structural element in the design when subject to the appropriate load combination at the fire limit state (FLS). As an example, in the case of an office structure, the Eurocodes (EN, 2002) use partial safety factors of 1.35 for permanent actions (dead loads) and 1.5 for variable actions (imposed loads) at ultimate limit state design (ULS). At the fire limit state, these factors become 1.00 and 0.50 for permanent and variable actions respectively. The limiting steel temperature is the temperature at which the steel member is capable of withstanding the applied load but will fail at higher temperatures.

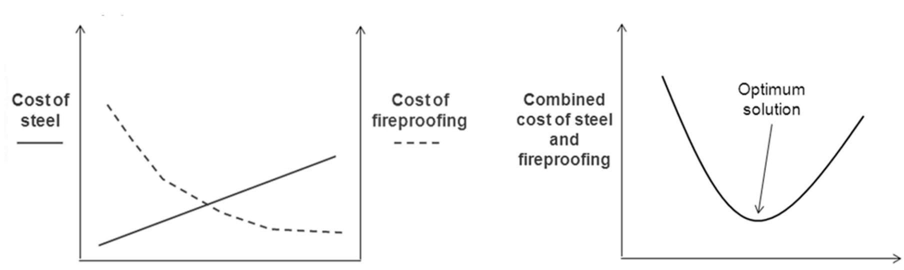


**Figure 3.** (Left) Illustration of the typical steel and fire proofing costs associated with an increase in the weight of a steel member, (right), combined cost of steel and fire proofing, showing optimum solution with respect to steel weight.

When the fire engineer and the structural engineer work together, the result is a quantified structural assessment that can form a detailed steelwork specification for fire protection requirements. This in turn is used by fire protection manufacturers to assess the quantity of fire protection material required on a project.

In some cases, a complete assessment of the structure in question using advanced numerical analysis may be undertaken (Lamont et al., 2006). This is a robust method to take into account the effects of whole frame behaviour including restrained thermal expansion and contraction forces and as the structure heats and cools.

### 2.1. Fire evolution in tall building design

For the purpose of structural design the fire has been schematized as a compartment fire and a worst case condition has been identified since the earlier studies of Thomas (Thomas, 1967) and later included in the methods described in (EN, 2002). These physically based descriptions of the fire generally result in conditions that can be deemed as less severe than the standard fires commonly recognised by local and international codes. Furthermore, methods to establish equivalence have been developed since the 1920s and applied to multiple designs (Ingberg, 1928). The characteristics of the compartment fire have been challenged many times (Stern-Gottfried et al., 2010) and alternative approaches have been proposed (Stern-Gottfried et al., 2012a, 2012b). These studies aim to establish the characteristics of a fire that could be used for structural performance assessment but invariably show that while the standard fire cannot describe important behaviour under cooling nor correct temperature gradients, for heating purposes, it is mostly conservative, especially in the case of steel structures. Given the conservative nature of the standard testing procedures, it is clear that there is much potential for gain if the fire was to be analysed in more detail.

Tall buildings represent a unique challenge to the standard fire definition. Not only are issues such as cooling of significant relevance, given the optimized nature of the structure, but also tall buildings offer the potential of unusual fire conditions. Beyond the conventional challenges to the "compartment fire" imposed by open floor plan offices, in a tall building, multiple storey atria allow for fires of potentially very long duration (Stern-Gotffried et al., 2012) that while less intense than a standard fire, incorporate long preheating periods that could result in thermal loads more significant than those of the standard fire (NIST, 2008). Furthermore, the use of multiple underground parking garages could lead to intense and long fires in areas where good structural performance might be critical. These fires currently remain un-quantified when it concerns their potential impact on the structure. A most important scenario that is unique to tall buildings and that has affected several buildings in the last few years is the multiple floor fire rapidly extending through façade systems. This scenario can result in a fire that can cover many floors simultaneously, thus resulting in a long duration fire that compromises a large portion of the structure. A well-known structural failure of this type is that of the Windsor Tower in Madrid (Parker, 2005). So, the potential gain associated to a comprehensive analysis of the fire is two-fold, it can result in a less over dimensioned definition of the required fire proofing, but also it can establish potential weaknesses of the structural design as well as the impact of scenarios not comprised within the concept of the compartment fire.

## 3. Fire Testing for Passive Fire Protection Materials

There are a number of internationally recognised fire safety codes and guidance documents in common use today including NFPA 101 (NFPA, 2006), the IBC (IBC, 2006), BS 9999 (BS, 2008) and numerous country specific standards. These documents typically indicate a period of fire resistance for the building in the form of a lookup table. Typically fire resistance periods for high-rise buildings are 120 minutes or 180 minutes depending on the type of structural member.

Where the design calls for fire protection to elements of structures, these documents stipulate that the materials protecting the members need to be tested in accordance with one of a number of fire test standards, including (UL, 2001; BS, 1987; EN, 2010 ; AS, 2005; GB, 2002):-

- UL 263 / ASTM E-119 – North, South and Central America (except for Brazil) and the Middle East
- BS 476: Parts 20-22 – UK, India, South-East Asia, Brazil and the Middle East
- EN 13381 – Europe
- AS 1530.4 – Australia
- GB 14907 – China

In the absence of an appraisal of a member's limiting temperature by a structural engineer, fire protection manufacturers and their trade associations have adopted conservative limiting temperatures (ASFP, 2010). These temperatures vary across the world and in some cases are dictated by the fire test standard. For example, BS-476 accepting countries use 550ºC for columns in compression, 620ºC for non-composite beams supporting concrete slabs or composite slabs and 520ºC for hollow sections (ASFP, 2010), in North America UL 263/ASTM E-119 (UL, 2011) uses 538ºC for columns and 593ºC for beams, while in parts of Europe the temperature is commonly 500ºC (EN, 2010). It has been acknowledged that these temperatures are acceptable for most circumstances but not always (ASFP, 2010).

Manufacturers of passive fire protection subject their products to a fire test package that comprises unloaded and loaded beams and columns with a range of protection thicknesses. In the UK and Europe (BS 476, EN 13381), the result of this test package is a data set that defines the required protection thickness for structural sections over a range of limiting steel temperatures, typically 350°C to 700°C in 50°C intervals.

## 4. Structural Analysis at Elevated Temperatures

Many structural design codes and guidance documents include 'fire resistant' design. In the UK the relevant standard is BS 5950 Part 8:2003 (BS, 2003) and in Europe the relevant codes are EN 1993-1-2:2005 (EN, 2005) for steel and EN 1994-1-2:2005 (EN, 2005) for composite steel and concrete design.

In BS 5950-8 and the Eurocodes methods are given for determining the thermal and mechanical response of the structure and evaluating the fire protection required, if any, to achieve the required performance. An important feature of the standards is that they use the concept of a variable steel temperature, i.e., they allow the limiting steel temperature to be evaluated. The following sections provide examples of the application of these methods.

A common method of assessment is to assess the degree of utilisation of a structural member in the fire limit state through a single element analysis. This approach considers beams or columns in isolation with conservative boundary support conditions and effectively reproduces the load-bearing scenario of a standard fire test as close as possible by calculation. Depending on the member subject to the analysis, a number of checks at elevated temperature may need to be accounted for including:-

- Tensile or buckling resistance for tension or compression members
- Moment resistance and shear capacity for beams
- Lateral torsional buckling resistance moment for beams

The following sections in this paper use the Eurocode approach to demonstrate the benefits of a single member analysis in the context of tall building design for standard beams and columns, concrete filled steel tubular columns and long span beams with web openings.

### 4.1. Typical beams and columns

The majority of structural members in buildings including high rise are typical serial sections, including I-sections, H-sections, W-shape and tubular columns and beams. With these members there is potential to make big economic savings in the volume of fire protection using simplified structural assessments.

A sensitivity study was presented by (Jowsey and Scott, 2012) in which the failure temperatures and intumescent coating thicknesses for 203×203×52 UC section in compression were calculated using Eurocode calculations in accordance with the UK National Annex. The steel strength, column height and degree of loading were all varied and comparisons for all cases were made back to the corresponding thickness of fire protection for the UK adopted temperature of 550°C for fire resistance periods of 90

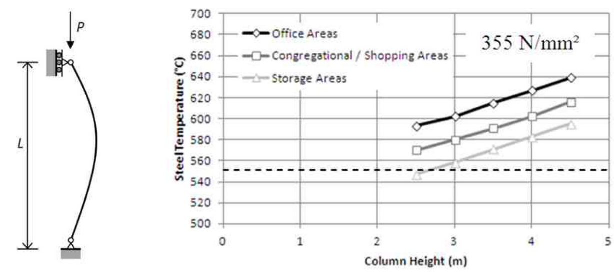


**Figure 4.** (Left) Column acting in compression. (Right) Limiting steel temperature variation for a fully loaded, Grade s355, 203×203×52 UC for a range of column heights and occupancy uses. The dashed black line represents the UK industry adopted temperature of 550°C. (Jowsey and Scott, 2012).

and 120 minutes. It was shown that a simple structural check using conservative assumptions could reduce the amount of fire protection by approximately 28% with potential further reductions being possible with more accurate loading information.

Figure 4 depicts the compression member and also shows the calculated limiting steel temperature for a range of column heights and occupancy types assuming a fully loaded column and grade s355 steel. It is evident that the UK industry adopted temperature of 550°C for a column is generally conservative.

In order to appreciate the benefit of calculating the limiting steel temperature, the corresponding thickness of fire protection is used to quantify the saving in fire protection material volume. Figure 5 shows the potential saving for the 203×203×52 UC section at a height of 4.5 m at 90 and 120 minutes fire resistance in comparison to assuming a conservative limiting steel temperature of 550°C.

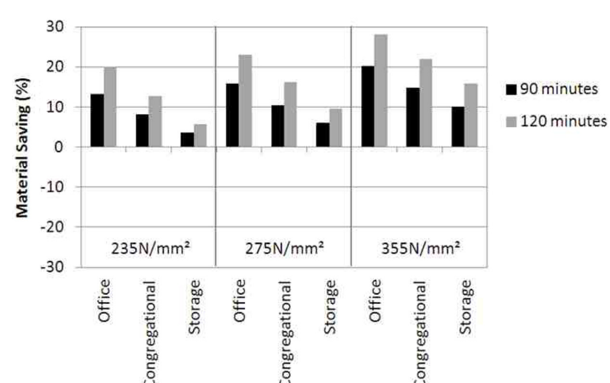


**Figure 5.** Fire protection material saving by calculating a limiting steel temperature for a fully loaded 4.5 m high 203 ×203×52 UC in comparison to an UK adopted temperature of 550°C. (Jowsey and Scott, 2012).

### 4.2. Concrete filled steel tube columns

In tall building design, one of the most efficient of all structural sections in resisting axial compression is a rein-

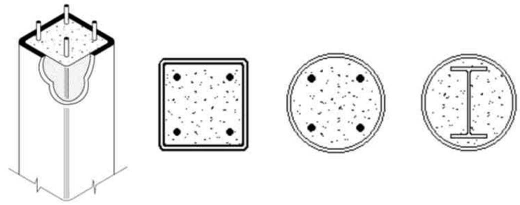

**Figure 6.** Concrete filled tube designs typically used in high-rise column design.

forced concrete filled steel tube. By filling hollow sections with concrete, a composite section is produced, as depicted in Fig. 6 with the result being an increase in the axial compressive resistance. Although methods are available to demonstrate that no applied fire protection may be necessary for these members (EN, 2005), for high periods of fire resistance, designs often fall outside the limits of minimum dimension and minimum reinforcement. Therefore the need to provide passive fire protection often remains.

In the absence of a structural assessment to determine the steel failure temperature, a passive fire protection manufacturer may use published conservative methodologies (Corus, 2002) that account for the concrete fill to reduce the section factor (ratio of heated perimeter to the cross-sectional area) of the section. This section factor is subsequently used to determine a thickness of fire protection based on testing of an unfilled hollow section using typical industry adopted limiting steel temperatures.

Methods however are available in design standards to analyse these members in the event of a fire (EN, 2005). When these are used to provide a limiting temperature to a fire protection manufacturer, an economical and optimised thickness can be determined.

The following example considers a concrete filled circular hollow section of overall diameter 139.7 mm and a wall thickness of 5 mm. The required fire resistance period for the project was 180 minutes. Using the conservative methodology described above, the effective section to account for the concrete infill was adopted and an intumescent coating thickness of 7.15 mm was required from a specific fire protection manufacturer.

As part of a value-engineering exercise, the authors undertook an analysis of the member using a numerical heat transfer assessment with the temperature dependent thermal material properties for steel and concrete (EN, 2005). A structural capacity check at the fire limit state was then carried out to ascertain the limiting steel in accordance with Annex H of Eurocode 4 (EN, 2005). The properties of the member were: length 3.25 m; pinned supports, grade s355 steel, concrete grade C40/50, 4 reinforcement bars at 8 mm diameter and 25 mm cover. The concrete filled column was conservatively assumed to be fully stressed at ambient, leading to an un-factored dead load of 310 kN and an imposed load of 403 kN at the ultimate limit state (ULS). Taking into consideration the fact that the building was an office, for the purpose of imposed load reduction at the fire limit state (FLS), the limiting steel temperature was determined to be 630ºC.

In conjunction with the passive fire protection manufacturer, an optimum thickness of intumescent coating was determined as 4.059 mm to ensure the steel tube was kept below 630ºC for the 180 minute fire resistance period. To validate the approach, a representative member of the same cross-sectional dimension with the applied intumescent coating was subjected to a fire test. Figure 7 shows the output of the test in terms of steel, concrete face and concrete centre temperature histories when exposed to the ISO-834 standard fire curve (ISO, 1994). It is apparent that at 180 minutes, the temperature of the steel is 610ºC and is therefore lower than the required temperature of

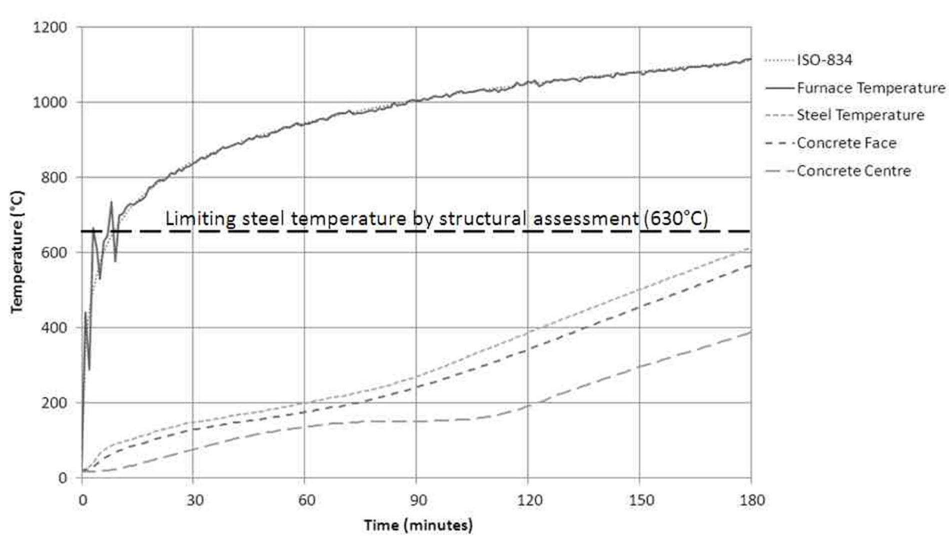


**Figure 7.** Temperature evolution for a fire test of a concrete filled steel tube protected with an intumescent coating with a set thickness to keep the steel temperature below 630ºC at 180 minutes.

630ºC to maintain structural stability.

The optimised and engineered coating thickness led to a 57% saving of fire protection material volume for this column, which was numerous in its occurrence throughout the building. This saving is in comparison to the conservative approach that would normally be adopted in the absence of a detailed structural fire assessment. It highlights the benefit of the approach and illustrates how close working with a passive fire protection manufacturer can lead to optimised solutions.

### 4.3. Beams with web openings

Beams with web-openings are becoming more and more popular with tall building design as they are a relatively lightweight and efficient method to achieve long spans to facilitate open plan floor layouts. They permit services to be passed through the openings rather than below the steel beam and are therefore efficient in terms of increasing the number of floors in high-rise buildings. These beams are commonly referred to as cellular beams in the UK and Europe or smart beams in North America.

The behaviour of these beams in fire has been the subject of much research over recent years both in terms of structural response (SCI, 2011) and thermal behaviour (EN, 2012). The openings in the web create a complex distribution of temperature on the web, specifically at the areas between adjacent openings (web-posts). It is important that such a temperature distribution is taken into consideration as part of structural assessment. Indeed, there are no industry adopted temperatures for these types of beam and a limiting temperature must be calculated and given to a fire protection manufacturer to determine an appropriate thickness of fire protection (ASFP, 2010). The structural response of these beams is more complex than solid-web beams. The distribution of force around the openings on the web means that in addition to bending and shear, the following failure modes are important to assess (SCI, 2011):-

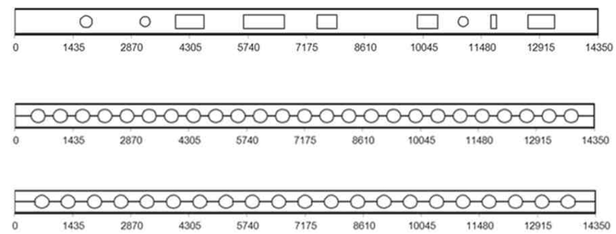


**Figure 8.** (Top) Initial beam design from the structural engineer, (middle) an alternate lightweight beam design from the steel fabricator, and (bottom) a lightweight design that is optimised for fire protection.

- Vertical shear check at openings
- Bending moment check at openings
- Vierendeel bending moment check at openings
- Web-post buckling
- Web-post bending
- Web-post horizontal shear

The following example considers three beam designs to span 14.35 m as shown in Fig. 8. The initial plate girder beam design was proposed by the structural engineer. The steel fabricator produced an alternate lightweight beam design, based on stock serial sections that reduced the cost of the steelwork; however this proved to be an overall more expensive solution as the fire protection costs were not factored into the design. Following a value engineering exercise by the authors, a final revised design was created that was optimised for fire with respect to steel cost, fire protection cost and application cost.

Figure 9 shows the relative cost and weight savings of the alternate and revised beam designs with respect to the original beam design. The fabricator's alternate solution resulted in a 9% saving in the steel weight, with a corresponding 9% saving in the cost of steel. However this design now had closely spaced web-openings creating hotter web-posts in the event of a fire – the result being an increased thickness of intumescent coating to the extent

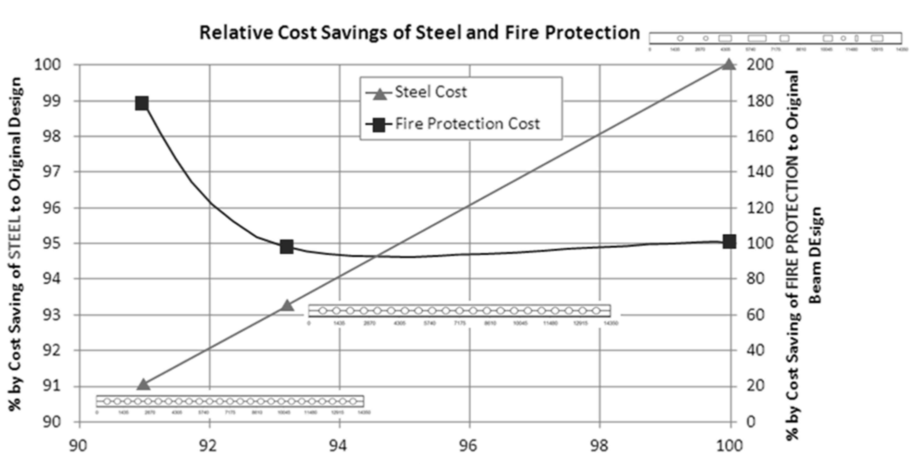


**Figure 9.** Representation of the three beam designs in terms of cost saving of steel and fire protection, both with respect to weight saving of steel.

that the application had gone from one coat of paint to two, with an associated increase in application cost. This has the effect of increasing the fire protection cost to 180% of the original beam. Following the value-engineering exercise with close attention paid to the fire protection, a revised structural design was proposed in which the openings were spaced further apart to create relatively cooler steel in the web-posts and the web thickness was increased to add 2 kg/m to the linear weight of the beam. This had the effect of providing enhanced structural capacity to combat the web-post buckling structural failure mode. The effect of these modifications was a beam that was 7% lighter and cheaper in terms of steelwork than the original design, but also approximately the same cost in terms of fire protection, i.e., it remained a single coat application of intumescent paint.

Figure 10 shows the combined cost of the steel and fire protection saving with respect to the original design for the alternate and revised beam designs. It is seen that the alternate design, although being 9% lighter in terms of steel weight, is 108% of the cost of the original beam design due the cost of the additional fire protection. The revised solution with an increased web thickness and opening spacing was shown to be 7% lighter in terms of steel weight, however it was 6% cheaper overall than the original beam design.

This example demonstrates that an optimum cost structural solution based on cost of steel and fire protection is possible and importantly it shows that the lightest structural design is not necessarily the cheapest solution. Given that these long-span structural members are commonly used in tall building design, the economic savings of analysing beams in this manner can be significant.

## 5. Conclusions

The issue of structural fire resistance is one that has been developing with a fast pace over recent years. Structural engineers are now starting to acknowledge and recognise that they can add significant value to their designs by considering fire as a load case in its own right. Structural design codes provide engineers with the opportunity to exploit the properties of structural steel to its maximum capacity in the fire limit state. If used effectively, benefits of structural fire engineering include significant cost and time savings on projects.

The following conclusions are noted: -

- A structural fire engineering assessment leads to informed performance, rather than assumed performance in the event of a fire.
- In the absence of a structural fire engineering assessment to determine a member's limiting steel temperature, fire protection manufacturers will assume generally conservative temperatures for the purpose of defining volumes of fire protection for a project.
- Simple approaches are available in published codes and standards to permit structural engineers to determine limiting steel temperatures for individual members.
- A truly optimised structure for fire should take into account the combined cost of steel, fire protection and application costs as a minimum.
- Significant space for further optimisation exists if a proper analysis of the fire is conducted, nevertheless, methods to do this, in spaces typical of tall buildings, are currently not well developed.

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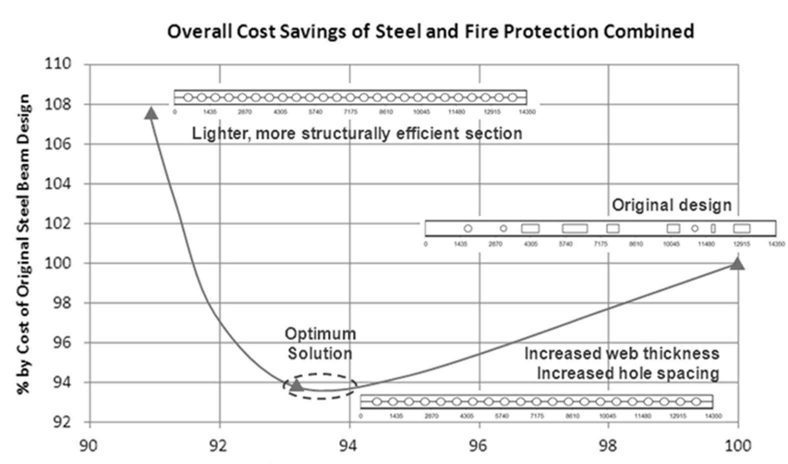


**Figure 10.** Overall cost savings of steel and fire protection combined for the three beam designs with respect to weight saving of steel.

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