Session 1

Case Studies

VISIBLE STEEL FOR A FIRE SAFE STRUCTURE
Case Study Office Building “Junghof”

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INTRODUCTION

In the centre of Frankfurt the office building “Junghof” had to be refurbished to make it attractive for modern use. When it was built in the 1950s five storeys were located around a central court. The new concept (*Fig. 1*) developed by the architects “Schneider und Schumacher” of Frankfurt added two storeys, raising the building above the critical height into the class of high-rise buildings. This led to additional requirements with regard to the fire-safety, mainly escape routes and fire resistance of structural elements.

Fig.1. View of the “Junghof” in Frankfurt

The building is located close to a district with many sky scrapers that rise up to 200 m. Because it is visible from most of these the architects invested much effort into the 5th façade. The roof was given a free form, clad with aluminium panels.

1 GEOMETRY

All four wings of the existing building have different depths. This led to varying geometric patterns in each corner. Spline-functions were used by the architects to define the form of the roof. These functions could not be used for further processing of the structural analyses and shop drawings, so an equivalent mathematical model was developed. To prepare buildable structural details it became necessary to develop plane glass surfaces and a roof structure with constant depth. This was done by defining a mathematical net for the outer surface. Relating to the net nodes, all additional design depths of the structure and dry ceiling were defined.

The system lines of the structural model for the calculation of the design forces were also taken from this net. This structural model then served as the basis for the final design drawings.

2 STRUCTURE

The concrete cores for the staircases in the four corners of the building had to be strengthened for the additional loads of the added storeys. Between these cores no additional loads were allowed to be transferred to the existing structure. Due to the difficulties related to improving the strength of the structure the added two storeys had to be light. Therefore, and because of the long span over the old structure, a steel construction was chosen.

Fig. 2. Primary structure of the 7th floor office space

A structure was designed that resembles an arch bridge (*Fig. 4a*). In fact only the sides located towards the court act like an arch (spanning 36 m) in structural terms. The outside structures are trusses with cantilevers showing top chords having the form of an arch. They have a total length of 57 and 60 m respectively. The cantilevers have a length of up to 12 m. These arches are made of circular steel sections (diameter: 355.6 mm, wall thickness: 12 to 20 mm). The tension element tying the arches horizontal bearing forces is an I-beam. All arches, tension elements, and bottom chords are fire protected with fire board.

Fig. 3. Roof beams

The need for a light structure combined with the architects goal to produce a transparent, open office space led to the use of composite slabs (depth = 160 mm, spanning nearly 2 m without temporary support) on composite beams spanning from outside wall to outside wall (maximum span = 7.5 m) without any interior columns (*Fig. 2*). This led to a common design of the 7th floor slab but to very odd shapes of the roof-beams on account of the shell-like roof (*Fig. 3*). Due to the use of a suspended ceiling it was possible to protect these beams with a cementitious coating that gave them 90 minutes fire resistance.

The design for heating and cooling required the use of material with high heat capacity in the roof since no air-conditioning was installed. Therefore slopes of up to 70° had to be formed in concrete. Up to 30° composite slabs with in-situ concrete were cast. A re-entrant composite profile (with a depth of 51 mm), needing no fire protection was used for the slab. It was flexible enough to follow the curve of the roof (*Fig. 3*). In bigger slopes pre-cast concrete elements were used (see also *Fig. 3* lower left area). They were bolted in three points to the steel structure. The joints were filled with fireproofing-material. Due to a careful planning process 90% of the roof-area was made of identical elements.

The 7th floor frames into a stringer that is suspended from the arch using circular sections (diameters of 40 and 56 mm) with fire-protection made of calcium-silicate shells.

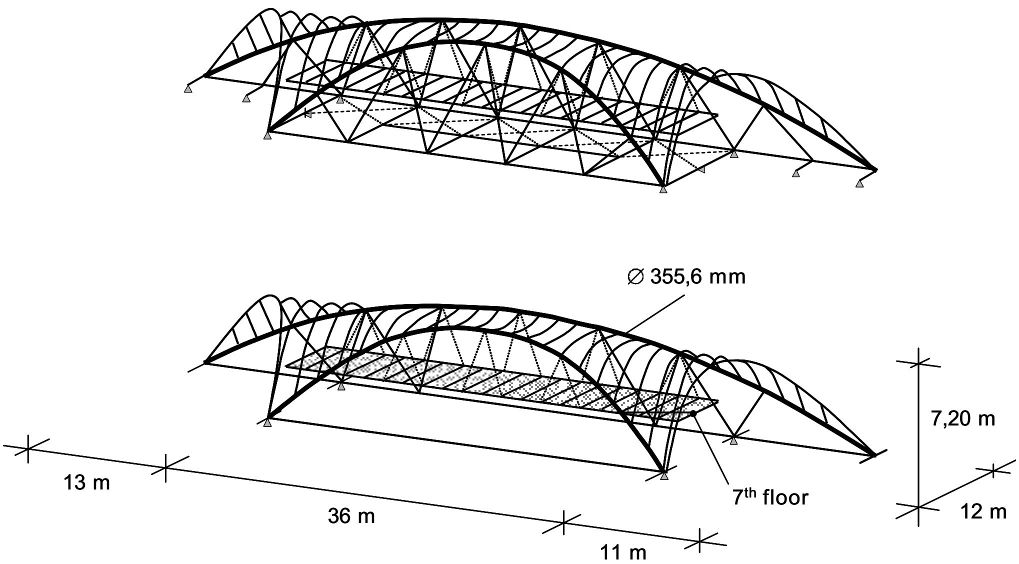

Fig. 4. Structural System
top: a) Serviceability state
bottom: b) System for fire design

3 FIRE DESIGN

A special feature of this building is the design of the ties that are part of the arch. They are designed without any fire protection. As shown in *Fig. 4a* a structural system was developed that in case of fire neglected most of the ties and the wind-loaded truss connecting the bottom chords (*Fig. 4b*). This system was analyzed under all dead and live loads. The wind loads were reduced, because it was reasonable to neglect the area of the windows. If the fire is strong enough to reduce the strength of the ties the windows will be broken.

Due to redistribution of the loads for some members the fire-design was the governing load case.

This design procedure, guided by references [1] to [9] not only saved cost but also reduced the visible width of the ties. In some load cases they have to carry compressive loads. Therefore it was not possible to reduce their diameter below 273 mm.

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STRUCTURAL FIRE ANALYSIS FOR A PERIMETER BRACED-FRAME STRUCTURE

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1 INTRODUCTION

This paper presents a detailed analysis of the structural performance of a braced perimeter tube in fire, based on a proposed landmark 63 storey office building in the City of London, UK. The aim of the analysis was to study the robustness of this iconic structure and to address any weaknesses in the proposed structural form. The analysis quantified the heating and cooling phase over the entire 3,000m^2 floor-plate and its effects on connections and structural elements, including the non-uniform perimeter bracing layout.

The outcome of the study presented in this paper allowed for an understanding of the strengths and weaknesses of structural design in terms of fire both for tall buildings in general and specific to this structure. The analyses have identified key performance issues which would not have been identified if the building were protected to typical Building Codes. This project has highlighted the need for more research into large diameter circular hollow sections filled with un-reinforced concrete and research into performance of connections.

2 THE BUILDING

The Pinnacle is 288m tall and comprises 3 basement levels and 63 upper floors (*Fig* 1). The building will be mainly occupied by offices. The structural system that is discussed in this paper is a braced perimeter tube, which is a system that is suited for very high rise buildings.

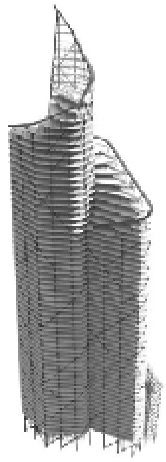

Fig 1: Structural representation of the building

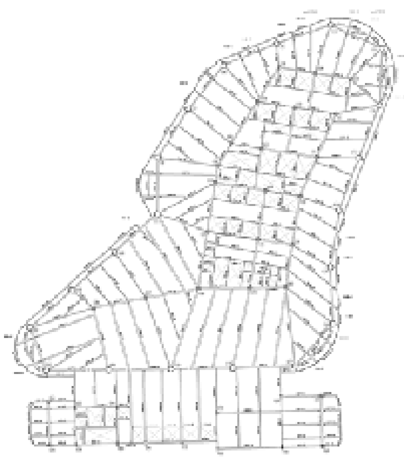

Fig 2: Typical beam layout for lower floor (Floor 5)

This building has several unique features. It has a highly irregular floor plan which has a beam layout that changes from floor to floor (*Fig* 2). The perimeter columns of the building are inclined due to the tapered elevation of the building. The layout of the perimeter diagonal braces is non-uniform as it has been optimised to resist the worst case wind condition. The braces connect to the perimeter columns at megaframe levels which are located at every third floor and act together with the beams to resist the wind and other horizontal forces.

3 STRUCUTRAL FIRE ANALYSIS

A series of non-linear structural fire analyses were performed by Arup Fire to assess the robustness of the building in a fire and to develop an engineered fire protection strategy for the structural steel members of the building. The engineered fire protection strategy incorporated unprotected secondary beams and reduced fire rating (90 minutes) to all structural frame elements, rather than relying on the prescriptive guidance defined by Building Regulations[1] which requires 2 hour fire protection to all structural elements. Three representative floors of the building (levels 5, 6 and 44) were selected to represent the structural fire response of the typical office floors and to address various aspects of the structure. The analyses of only one of the three floors (level 5) are presented here. There were several challenges in undertaking the structural fire analysis for this project:

- The architects expressed their desire to have large, clear spans with a minimum number of internal columns to provide flexibility for the building tenants.
- To minimise the inter-storey height, cellular beams with composite steel-concrete trapezoidal floor decks were to be used. The cellular beams allowed the building services to be passed through the beam webs while the trapezoidal floor system reduced overall building weight.
- Unreinforced concrete filled steel tube columns were also proposed to be used around the perimeter of the building.
- Other aspects that had to be considered in the analyses were the increased temperatures of the shear studs due to the proposed trapezoidal decking system (without fire protection to fill the voids) and any effects of partial composite action on structural fire performance.
- Web-post buckling of the protected cellular beams had to be mitigated.

3.1 Acceptance criteria for analyses

The main acceptance criteria for the analyses considered maintaining stability and compartmentation throughout the duration of the design fire, specifically:

- Columns to maintain their load carrying capacity and no runaway deflections of the floor system to occur for the duration of the fire scenarios
- Strains in the concrete floor slab, specifically at connections, are within acceptable limits.

3.2 Design Fire

The parametric fire curve described in Eurocode 1: Part 1-2 (BS EN 1991-1-2:2002) [2] was used to determine a credible natural, office design fire (*Fig* 3). This design fire was applied throughout the entire floor plate, assuming simultaneous uniform heating throughout a single floor. Multi-floor fires were not considered. In determining the worst case design fire, various amounts of ventilation were considered, ranging from 25% to 100% to consider different amounts of glass breakage in the fire. The fire with 25% ventilation was chosen and agreed with the Approving Authorities (City of London) as it was considered to have the greatest impact on the protected structural elements and to be the most conservative design fire.

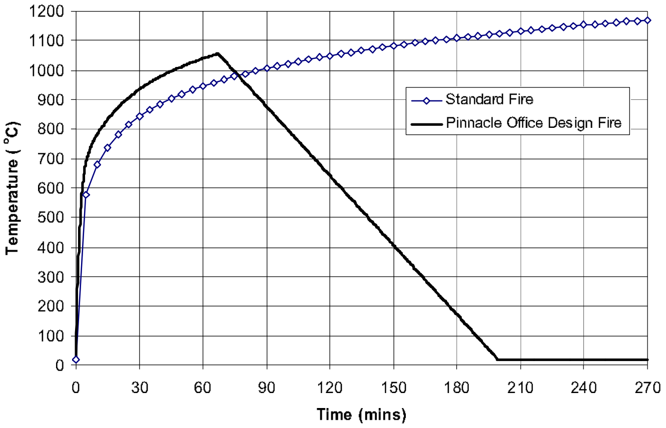

Fig 3: Comparison of design fire with the Standard Fire

4 MODELLING OF THE GLOBAL STABILITY SYSTEM

The program used for the analysis of The Pinnacle is the ABAQUS [3] Finite Element Analysis (FEA) software, incorporating the non-linear temperature dependant material properties based on the Eurocodes [4],[5],[6].

Fig 4 shows the 3D structural model to simulate a fire on level 4, which heats the Level 5 floor and the megaframe levels between Levels 2 to 8. The unheated floors (Levels 3, 4, 6, 7 and 8) were assumed to behave as and modelled as rigid diaphragms because the in-plane stiffness of the unheated floors was considered to be significantly greater than the bending resistance of the columns. This was modelled using kinematic rigid links which tie all the columns to a central reference point, which represents the centre of gravity of the building.

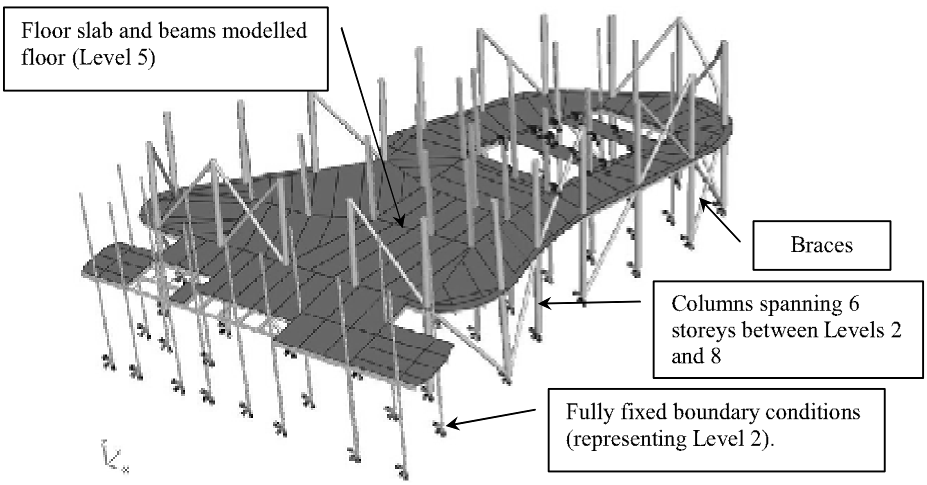

Fig 4: Level 5 global finite element model showing boundary conditions and mega-frame structure between floors 2 to 8. (The unheated floor diaphragms representing the unheated floors are not shown)

Many structural fire analyses in commercial applications involve buildings with regular beams and column layouts which can be analysed using simplified representative portions of part of a floor to represent its overall response. Also, the lateral stability system usually consists of a reinforced concrete core which is assumed to be relatively unaffected by fire and as such the building is assumed to be laterally restrained. However, for The Pinnacle, the entire 2800m^2 floor plate had to be modelled because of the irregular floor shape, the irregular bracing and because the entire perimeter frame has to resist the lateral loads due to wind at the fire limit state. The assumption that the lateral stability system was not affected by fire could not be applied for this building.

In the model, all columns and bracing in the model were extended to the next megaframe level above and below the analysed floor to accurately simulate the column and bracing boundary conditions.

To minimise the loss of lettable area caused by intrusion of the braces into office spaces, the braces have to change direction where they touch the intermediate levels between megaframe levels. This is structurally less efficient and can cause significant forces to be passed into the intermediate floors. This had to be modelled and monitored in the structural fire analysis to ensure that the forces do not cause failure of the beams and floor slab.

The applied live, dead and wind loads were factored in accordance with BS 5950 Part 8 (2003) [7]. Lateral wind forces which apply the most significant loading onto the structure were included by applying the resultant forces on the columns and the braces.

Four node shell elements were used to model the slab. Two node linear beam elements (B31) were used for all beams, columns and braces. The reinforcing steel in the slab was modelled based on a smeared model, assuming a thin sheet of steel of equivalent area to the specified reinforcing bars.

The modelling of composite columns on a large global finite element model was challenging although there has been research into modelling the performance of concrete filled tubes [8]. Two different beam finite element sections were linked together to model the single concrete filled composite steel section. One of the beam sections had a circular hollow profile with steel properties to represent the steel tube while the other had a solid circular section with concrete properties. The concrete used for infill was C80 high strength concrete. The concrete was conservatively modelled as having uniform temperatures, with temperatures 20% higher than the steel tubes.

4.1 Modelling of web penetrations and partial composite action in beams

The primary and secondary beams are 625mm deep cellular beams and have circular web penetrations to allow mechanical and electrical services to be passed through. In the global structural models, the beams were modelled with the full web thickness of the beams and without web openings. The beam finite elements cannot model the web penetrations and any effects on the beam behaviour in fire due to these penetrations.

A separate analysis of a detailed subassembly of part of the structure using only shell finite elements was undertaken to assess and capture any fire related phenomena on the web penetrations, such as web-post buckling of the beams, and partial shear composite action between the slab and beam.

4.2 Beam connections

Typically, beam connections are modelled as either fully pinned or fully fixed, for simplicity. For The Pinnacle, the beam connection capacities were available from the structural engineers and they were specified with temperature dependent capacity limits (shear, axial and/or moment), based on the Eurocode strength reduction curves [5]. This allows the transfer and redistribution of forces due to local connection weaknesses. The connectors can continue to deform once they reach their maximum capacity, but the limiting force (in tension, bending or shear) is not surpassed.

5 MODELLING RESULTS

To make the models run and cope numerically with the large number of variables and the highly complex nature of the structure in fire required creative application and integration of software and hardware. The analyses for Level 5 showed that global stability was maintained throughout the entire fire duration and there were no runaway deflections in the slab or beams (Fig 5). There was also no failure of the columns due to buckling. The largest mid-span deflections (beam 28) and the adjacent parallel protected secondary beam are shown in *Fig* 6. The floor system supported by unprotected beams performed well due to tensile membrane action. The unprotected beams did not suffer runaway failure because of catenary action while the protected beams were able to resist the loads by bending action. The detailed submodel of part of level 5 (*Fig* 7) showed that although the unprotected secondary beams suffered large lateral distortions, the protected primary beams with the 90 minute fire protection did not buckle and were relatively undeformed.

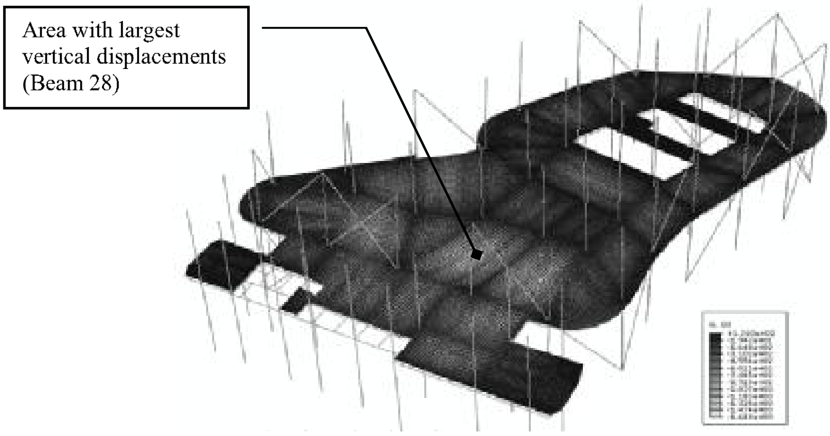

Fig 5: Vertical displacements at end of the analysis for Level 5.

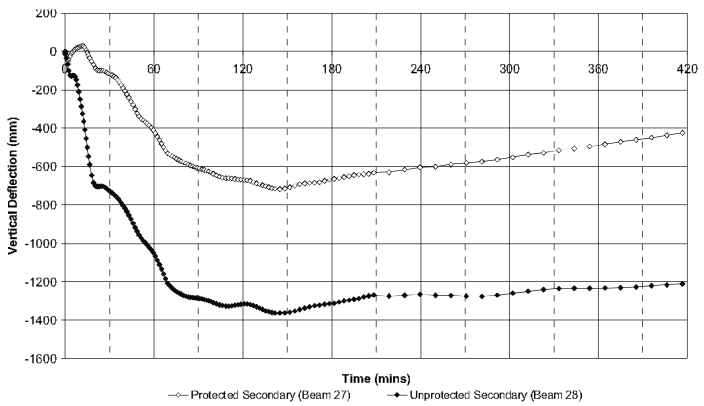

Fig 6: Mid-span deflections with time for selected worst-case beams.

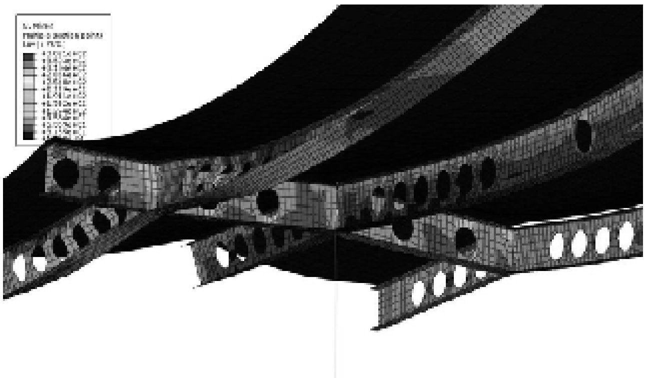

Fig 7: Submodel showing detailed deformation of beams on level 5.

The bracing elements had a significant influence on the deformation and forces within the structure. The analyses showed that the expansion of the protected braces due to heating can impose significant axial forces on the columns. The heating of the braces in a non-uniform bracing arrangement can influence the deformation of the floor plate due to thermal expansion.

6 CONCLUSIONS

This paper describes the 3D structural fire analysis for a complex high rise structural system to check its robustness when exposed to a realistic fire. The analyses showed that the structure retained its stability throughout the entire duration of the design fire. The analysis showed that a engineered fire protection strategy incorporating a reduced fire rating and also removal of fire protection to the secondary beams could be safely applied. This project has also highlighted the need for more research into un-reinforced concrete filled hollow steel sections.

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SIMULATION AND STUDY OF NATURAL FIRE IN A WIDE-FRAMED MULTIPURPOSE HALL WITH STEEL ROOF TRUSS

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INTRODUCTION

As a part of an extensive long-term plan for the development of the Helsinki Fair Centre a new 14500 m^2 multipurpose hall will be built in Helsinki, Finland. The design of the hall was carried out in co-operation between Finnish and German architects and it is intended to be able to host both exhibitions and indoor sporting events with up to 6000 spectators. In Figure 1, the new multipurpose hall can be seen to the left.

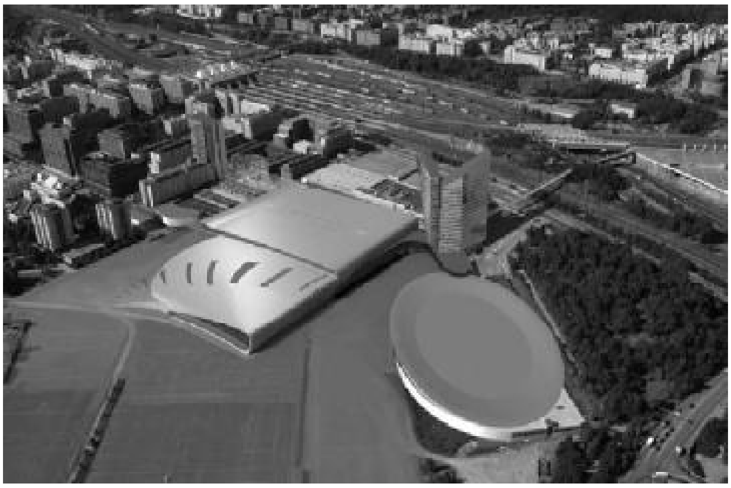

Fig. 1. Air photo of the architects´ suggestion for the Helsinki Fair Centre

The Finnish Building Code allows the fire safety design of a building to be performed according to either the prescriptive regulations or the natural fire safety concept, NFSC. In this study the NFSC was used to do a performance-based structural fire safety design of the steel roof trusses in the above described multipurpose hall. This made it possible to estimate whether the roof trusses can be safe in the case of fire without passive fire protection, such as intumescent painting, or not.

1 STRUCTURE DESCRIPTION

1.1 General

The main frame of the hall is made up of reinforced concrete columns and three dimensional steel roof trusses. See Figure 2 for a view of the frame. The frame spacing is 9,0 m, except in the middle of the hall where it is 13,05 m. The span of the roof trusses is 78 m with a splice in the middle of the span and the total number of trusses is 17. The free height inside the hall varies from 10 m to 16 m. The total length of the building is just over 170 m and the total width is about 88 m.
The height of the steel roof trusses varies between 4,5 m and 7 m, the bottom width of the truss is about 1,3 m and the top width is 3,0 m. The trusses are made up of structural steel hollow sections of varying dimensions with a steel quality of S355.

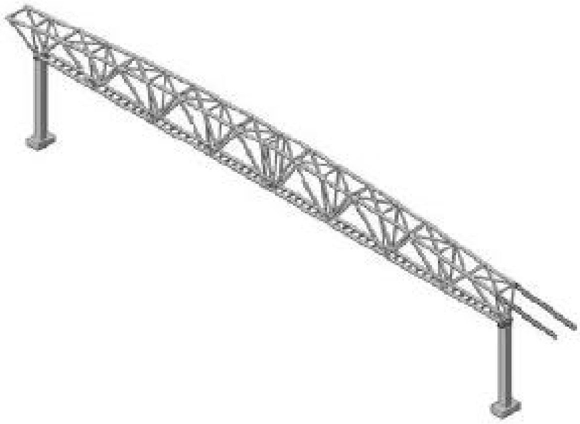

Fig. 2. Main frame

The roof structure will consist of wooden elements with mineral wool insulation supported by the roof trusses. A large part of the exterior walls will be made of glass, which is assumed to break when the temperature reaches 200 ˚C. The rest of the exterior walls will be made of steel sheets with mineral wool insulation and concrete sandwich structures.
The new hall will form one single fire compartment together with an existing hall at the Helsinki Fair Centre. Hence the total area of the fire compartment will be in the order of 33000 m^2.

1.2 Fire resistance requirement

In this study focus was only on the first part of the essential requirement for the limitation of fire risks according to the Construction Products Directive 89/106/EEC, i.e. "The load bearing resistance of the construction can be assumed for a specified period of time" [1]. The required time period in this case study was 60 minutes.

2 DESIGN FIRES

Two prescribed design fires were used in this study. They were chosen from the project's performance-based fire safety design report [2], where a rough risk analysis has been made and several different fires have been considered, and a report by Hietaniemi [3]. The choice of the design fires from these reports was based on the possible severity of their effect on the steel roof trusses.

2.1 Spectator stand fire

The spectator stand fire represents the case when the multipurpose hall is used for e.g. indoor sport events or concerts. As spectator stands can be placed also above floor level the design fire is closer to the roof than a fire on floor level, hence posing a larger threat to the load-carrying structure.
According to Hietaniemi [3] the maximum rate of heat release, RHR, of the seat material for a spectator stand fire can be assumed to be 2000 kW/m^2, giving a maximum RHR of the spectator stand of 1500 kW/m^2, while the total fire load being 510 MJ/m^2. The studied spectator stand section measured 8,0 x 13,6 m^2 with a height of 3 m. It was placed 5,2 m above floor level in the lower part of the hall, leaving only about 2 m of free height to the bottom chord of the roof truss. The whole section was assumed to be on fire, giving a maximum RHR of 163,2 MW. In Figure 3 the RHR curve for the spectator stand design fire is presented.

2.2 Exhibition stand fire

The exhibition stand fire represents one of the main purposes of use where the fire load can be very high. Also this design fire was based on Hietaniemi's report [3], a 10 x 10 m^2 exhibition stand, made of burnable materials, for small motor vehicles, e.g. motorcycles or ATV's, placed on floor level in the lower part of the hall just below a roof truss. The maximum RHR of the design fire was 53 MW, the total fire load was 1720 MJ/m^2 and the height of the stand was assumed to be 2 m. The RHR curve for the exhibition stand fire can be seen in Figure 4.

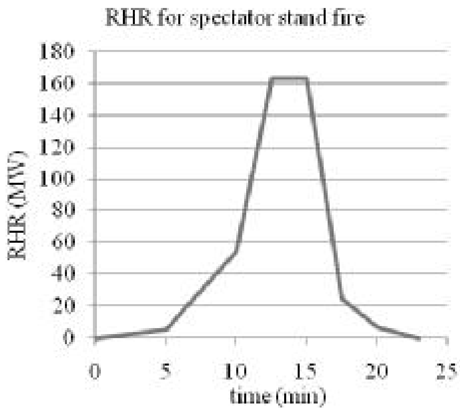

Fig. 3. RHR curve for spectator stand fire

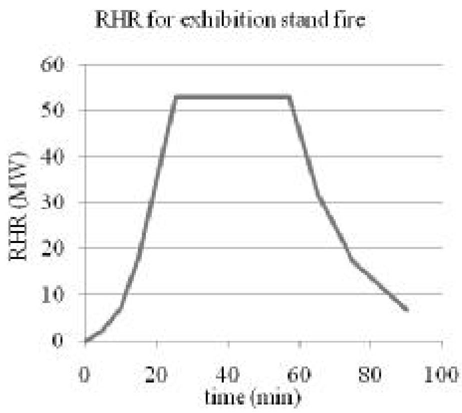

Fig. 4. RHR curve for exhibition stand fire

2.3 Sprinkler system fail

As the sprinkler system usually is considered to be one of the most effective and reliable active fire protection systems in a building, a total sprinkler system fail was not considered in this study. However, a partial sprinkler system fail, where two nozzles above the design fire were inoperative, was considered in both the design fires.

3 FDS MODEL CREATION

3.1 General

The design fires were simulated with Fire Dynamics Simulator, FDS, version 5.2 and the input files were made with Pyrosim. The whole multipurpose hall was modeled in Pyrosim with a cell size ranging from 0,2 x 0,2 x 0,2 m^3 to 0,8 x 0,8 x 0,8 m^3, hence the total number of cells was kept at about 2 millions.

3.2 Sprinkler system

The model was equipped with an automatic sprinkler system that activated when the temperature of the nozzle reached 74 ˚C. The sprinklers' effect on the gas temperature and the combustion occurring in the gas phase was taken into account in the simulation. The sprinklers' suppressing effect on the design fire was however not taken into account in the two original simulations, as there was no way of determining how large the effect could be without carrying out real fire tests. In a second additional simulation of the spectator stand fire, the effect was however taken into account according to a method described in Hietaniemi's report [3]. According to this method the RHR is only allowed to double from the value it has when the sprinkler system is activated. As the model was so large, the time when 20 nozzles had been activated in the original simulation, i.e. at approximately 7 minutes into the simulation, was used as the sprinkler activation point. At that point the RHR was about 28 MW and was hence allowed to grow to 56 MW.

3.3 Smoke exhaust system

The model was also equipped with a smoke exhaust system. The hall was divided into different smoke sections so that one section was about 2400 m^2, assuming that that the system could be active only in two sections at the same time. The time at which the smoke exhaust system was activated in the spectator stand fire was assumed to be 400 s and in the exhibition stand fire 600 s. These times were approximated based on simulation tests of the sprinkler activation times, assuming that the smoke exhaust system would be activated roughly at the same time as the sprinklers.

3.4 Data measured

The most important data measured during the fire simulation, from the structural fire safety design's point of view, was the adiabatic surface temperature every 5 m of the bottom chord of the steel roof trusses situated above and near to the fire. The adiabatic surface temperature is the temperature that the bottom chord "sees" and is the quantity that is representative of the heat flux to the solid surface [4]. This temperature was used to calculate the temperature of the steel cross section.
The gas temperature of the building was also measured at several different heights and points to get a picture of the total temperature development in the building as a function of time.

4 SIMULATION OF FIRE

4.1 Spectator stand fire 1

The original spectator stand fire simulation was ended at 1400 s, as the fire only lasted 1380 s. The RHR reached a maximum value of 167 MW during the simulation, i.e. very close to the intended value of 163,2 MW. The first sprinkler nozzle activated a bit sooner than expected, already at 308 s, being one of the sprinklers above the fire. In total 251 out of the 310 functional sprinkler nozzles in the model were activated during the simulation.
The temperature inside the hall remained quite low in general, except of course over and near to the fire. Close to the roof the temperature reached about 65 ˚C, while it at 5 m above floor level remained just above 20 ˚C.
The measured adiabatic surface temperature of the bottom chord just above the fire is plotted in Figure 5 and this was also the measurement used to calculate the temperature of the steel.

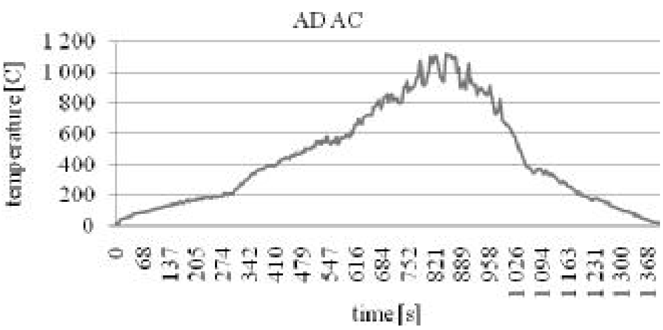

Fig. 5. Adiabatic surface temperature of the bottom chord above the spectator stand fire 1

4.2 Spectator stand fire 2

In the additional simulation of the spectator stand fire, where the sprinklers' suppressing effect on the design fire was taken into account, the RHR reached a maximum value of 56 MW.
As the sprinklers were removed from the model in order to speed up the simulation, the only measured data taken into consideration was the adiabatic surface temperature of the bottom chord just above the fire. This temperature is plotted in Figure 6.

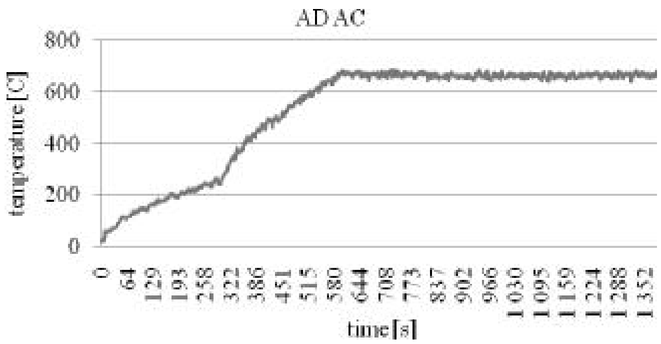

Fig. 6. Adiabatic surface temperature of the bottom chord above the spectator stand fire 2

4.3 Exhibition stand fire

The exhibition stand fire was ended at 3670 s, as there was no need to study the fire situation beyond one hour. The maximum RHR measured during the simulation was just over 53 MW, i.e. almost exactly the intended value. The first sprinkler nozzle activated a bit later than expected, at 668 s. In total only 148 out of the 310 functional sprinkler nozzles were activated.
The temperature close to the roof reached circa 60 ˚C while it at 5 m above the floor level only increased a few degrees above the original temperature of 20 ˚C, again except close to and over the fire.
The adiabatic surface temperature of the bottom chord above the fire is plotted in Figure 7.

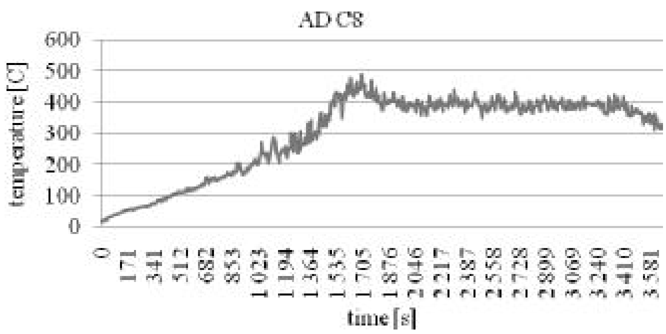

Fig. 7. Adiabatic surface temperature of the bottom chord above the exhibition stand fire

5 FIRE SAFETY DESIGN OF STEEL ROOF TRUSS

The temperature development of the unprotected steel members was calculated according to Eurocode 1 and 3 just above the fire. The temperature was assumed to be vertically equivalent.

5.1 Steel temperature

In the original spectator stand fire the maximum temperature of the bottom chord with a wall thickness of 10 mm was established to be 727 ˚C whereas it was 897 ˚C for the diagonals with a wall thickness of 5 mm.
In the additional simulation of the spectator stand fire the temperature of the bottom chord reached 614 ˚C whereas the diagonals reached a temperature of 661 ˚C.
In the case of the exhibition stand fire the maximum temperature of the bottom chord was 387 ˚C and of the diagonals 396 ˚C, i.e. almost the same temperature was reached in all steel sections, independent of wall thickness.

5.2 Structural analysis

A FEM-model of the roof truss was made in Robot Millennium 21. In the model the truss was subjected to snow load, self-weight of the roof structure, self-weight of equipment, such as lighting and ventilation ducts, hanged to the bottom chord of the truss and of course the self-weight of the truss itself. The temperature of the truss was assumed to be uniform in order to simplify the calculations and to avoid having to take the effect of heat conduction inside the roof truss into consideration. The effective yield strength and the modulus of elasticity of the steel were changed to correspond to the values at the different elevated temperatures.
With the help of the FEM-model the critical temperature of the roof truss could be established to be 590 ˚C. At this temperature the highest degree of utilization was 0,94 and took place in the diagonals in compression closest to the ends of the truss. The deflection was established to be 400 mm in the middle of the truss, without taking the pre-camber of 100 mm into consideration. Hence the actual deflection would be in the order of 300 mm, which equals the length of the truss, 78 m, divided by 260.

6 CONCLUSIONS

By comparing the temperature of the steel reached in the different design fires to the critical temperature of the truss, it could be established that the truss well could withstand the exhibition stand fire without any fire protection. However, in the case of the spectator stand fire the temperature proved to be too high for the unprotected truss to endure the fire, even though the temperature did not rise that much above the critical temperature when taking the sprinklers' suppressing effect on the fire into account.

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AN INNOVATIVE APPROACH TO DESIGN FIRES FOR STRUCTURAL ANALYSIS OF NON-CONVENTIONAL BUILDINGS
A Case Study

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INTRODUCTION

Structural fire safety analysis is a fast developing discipline within the overall fire safety community. There is substantial research being done in the mechanical response of buildings to thermal loadings. The results of this research are being applied commercially in the design of real buildings in numerous countries globally.

While a great deal of effort is being placed in the analysis of structures, there is a dearth of research on determining appropriate thermal inputs for such a structural analysis. Much of the structural fire safety work being carried out is based on one of two traditional methods of determining the fire environment; the standard temperature-time curve (which has its origins in the late 19th century [1]) or parametric temperature-time curves such as that specified in Eurocode 1 [2].

While both of these methods have great merits and represented breakthroughs in the discipline at their times of adoption, they have inherent limitations with regards to their range of applicability [3]. For example, Eurocode 1 states that the design equations are only valid for compartments with floor areas up to 500 m^2 and heights up to 4 m, the enclosure must have no openings through the ceiling, and the compartment linings are also restricted to a thermal inertia between 1000 and 2200 $J/m^2s^{½}K$, which means that highly conductive linings such as glass facades and highly insulated materials can not be taken into account. As a result, common features in modern construction like large enclosures, high ceilings, atria, large open spaces, multiple floors connected by voids, and glass façades are excluded from the range of applicability of the current methodologies. These limitations, which are largely associated with the physical size and geometric features of the experimental compartments on which the methods are based, ought to be carefully considered when the methods are applied to an engineering design. This is particularly relevant given the large floor plates and complicated architecture of modern buildings.

Another limitation of the existing methods is that they assume only uniform temperature conditions throughout the whole floor of the compartment. A fire that would cause these types of conditions burns uniformly within the enclosure and generates high temperatures for a relatively short duration. This is opposed to a travelling fire that burns locally but spreads through the enclosure with time, generating lower temperatures for longer times. Buchanan [4] notes that post-flashover fires in open plan offices are unlikely to burn throughout the whole space at once. Real, large fires that have lead to structural failure, such as those in the World Trade Center towers 1, 2 [5] and 7 [6] in September 2001, the Windsor Tower in Madrid, Spain in February 2005 [7] and the Faculty of Architecture building at TU Delft in the Netherlands in May 2008 [8] were all observed to travel across floor plates, and vertically between floors, rather than burn uniformly for their duration. Travelling fires have also been observed experimentally in compartments with non-uniform ventilation [9, 10]. Clifton [11] has developed a methodology to examine a spreading fire within a floor plate, but there is a lack of experimental data to validate this approach [4].

Therefore a methodology is being developed that allows for a wide range of possible fires, including both uniform burning and travelling fires, by considering the fire dynamics within a given building [3]. This new methodology also facilitates the collaboration between fire safety engineers and structural fire engineers, which is an identified need within the structural fire community [12], to jointly determine the most challenging fire scenarios for a structure.

This paper presents a case study of the application of this new methodology to a large, non-conventional structure.

1 NEW METHODOLOGY

The key aspect of the new methodology being developed is to characterise the thermal environment for structural analysis accounting for the fire dynamics specific to the building, including a wide range of possible fires. In order to achieve this a tool to capture the spatial and temporal changes of the fire-induced thermal field is selected. This tool is then applied to a particular floor of the building accounting for a family of fires that range between one that burns in a small area and travels across the floor plate for a long duration and one that is well distributed across the whole floor plate but burns for a short duration.

1.1 Temperature Field

The methodology divides the effect of a fire on structural elements into the near field and the far field [3]. The near field is when a structural element is exposed directly to the flames of the fire and the far field is when it is exposed to the hot gases, i.e. the smoke layer, away from the flames, as shown in *Fig. 1*. This division of the thermal field allows the methodology to overcome a well-known inaccuracy of fire modelling; calculation of the flame temperature.

A ceiling jet correlation, as developed by Alpert [13] and given below in *Eq. (1)*, was selected for this case study to determine the temperature field as a function of distance from the fire. The use of such a correlation is deemed appropriate if the floor area is large and the smoke layer is thin relative to the floor to ceiling height.

$$T_{\max} - T_{\infty} = \frac{5.38\left(\dot{Q}/r\right)^{2/3}}{H} \tag{1}$$

where T_{max} is the maximum ceiling jet temperature (K)
T_{∞} is the ambient temperature (K)
$\dot{Q}$ is the total heat release rate (kW)
r is the distance from the centre of the fire (m)
H is the floor to ceiling height (m)

Note that while Alpert gives a piecewise equation for maximum ceiling jet temperatures to describe the near field ($r/H \leq 0.18$) and far field ($r/H > 0.18$) temperatures, only the far field equation is utilised in the present case study as the near field temperature is assumed to be the flame temperature, as described in Section 1.3.

The ceiling jet correlation characterises the spatial variation of the temperature field in only a one dimensional manner, as it has been assumed that the fire area always extends between the

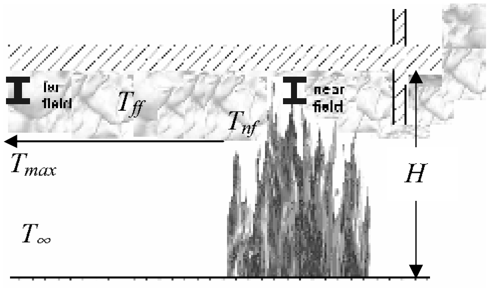

Fig. 1. Illustration of near and far fields

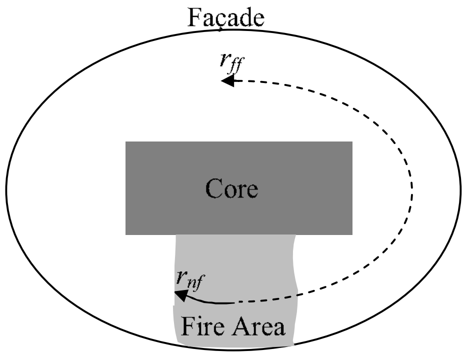

Fig. 2. Plan view of a possible fire path

core and façade of the building (see *Fig. 2*). The correlation was chosen to provide a straightforward description of the temperature field that is qualitatively sufficient to progress the development of the methodology. It is noted that alternative methods of characterising the temperature field, including computational fluid dynamics, can be utilised instead [3].

1.2 Family of Fires

A family, or set, of fires that covers the range of possible fires, both travelling and uniformly burning, needs to be selected as an input for the temperature field. To do this it was assumed that each fire in the family would burn over a surface, A_b, which is a percentage of the total floor area, A_t, of the building, ranging from 1% to 100%. The burning area of the fire that is equal to 100% of the floor area is a well distributed fire. All other burning areas represent travelling fires of different sizes.

It is assumed that there is a uniform fuel load across the fire path and the fire will burn at a constant heat release per unit area typical of the building load under study. Thus the burning time can be calculated by *Eq. (2)*.

$$t_b = \frac{q_f}{\dot{Q}''} \tag{2}$$

where t_b is the burning time (s)
q_f is the fuel load density (MJ/m^2)
$\dot{Q}''$ is the heat release rate per unit area (MW/m^2)

Note that the burning time is independent of the burning area [3]. Thus the 100% burning area and the 1% burning area will both consume all of the fuel in the same time t_b. However, a travelling fire moves from one burning area to the next so that the total burning duration, t_{total}, across the floor plate is extended. This time is given in *Eq. (3)*.

$$t_{total} = \frac{t_b}{A_b / A_t} \tag{3}$$

This means that there is a longer total burning duration for smaller burning areas.

1.3 Near Field vs. Far Field

In the case of the 100% burning area, all of the structure will experience near field (flame) conditions for the total burning duration (which is equal to the burning time, t_b). However, for the travelling fire cases, any one structural element will feel far field (non-flame) conditions for the majority of the total burning duration and near field (flame) conditions for the burning time as the fire burns locally to the element. The time one element experiences far field conditions prior to the arrival of the flame is defined as t_{pre} and the time the element experiences far field conditions after the departure of the flame is defined as t_{post}. *Fig. 1* illustrates the difference between the near field and far field.

The near field temperature, T_{nf}, is taken here as the flame temperature, which for the accuracy levels required in structural fire analysis, is more or less constant and approximately 1200°C to 1300°C for a typical office fire [14]. The far field conditions vary as a function of distance away from the fire. However, it is desirable to express the results in simple terms but without loss of generality in order to be of valuable engineering use. Thus, the far field is reduced to a single characteristic temperature, T_{ff}, which keeps the amount of information passed to the structural analysis manageable. To do this, the far field temperature is taken as the fourth-power average of T_{max} (to favour high temperatures in a bias towards radiation heat transfer and worst case conditions) over the distance between the end of the near field, r_{nf}, and the end of the far field, r_{ff}. This average is calculated by *Eq. (4)*.

$$T_{ff} = \frac{\left[\int_{r_{nf}}^{r_{ff}} (T_{\max})^4 \, dr\right]^{1/4}}{\left(r_{ff} - r_{nf}\right)^{1/4}} \tag{4}$$

Once the far field temperature is determined for a given fire size, the temperature time history of a point can be described, as shown in *Fig. 3*. Determining both t_{pre} and t_{post} is dependant on the path of the fire and the exact position being examined. However, it is not possible to establish a fire's path of travel *a priori* to a real incident; therefore assumptions must be made for worst case conditions. For the present case study, it is assumed that the fire will travel in a ring around the building core in one direction only, as illustrated in *Fig. 2*. Clearly other paths of fire travel are possible and the sensitivity of this parameter on the structural response will need to be explored as the methodology is further developed.

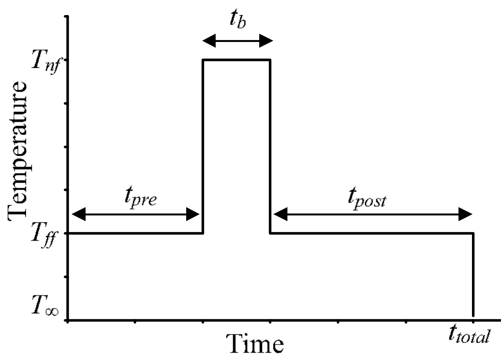

Fig. 3. Temperature-time curve at a general location of the ceiling for a travelling fire

Fig. 4. Mumbai C70 by James Law Cybertecture

2 THE CASE STUDY

2.1 The Building

The above methodology was applied to a real building, currently being designed. The building, Mumbai C70 by James Law Cybertecture shown in *Fig. 4*, is a large, modern, architecturally complex building in which a traditional temperature-time curve (standard or parametric) does not fully describe the range of possible fire dynamics.

The building has 13 storeys and is approximately 60 m tall. The building has an external diagrid megaframe consisting of hollow structural steel members, designed to carry wind loads and a proportion of the gravity load. The building is also provided with an internal reinforced concrete core system designed to carry gravity load. A hat truss is proposed at the top of the building. *Fig. 2* shows a generic plan view of a single floor. The exact shape and area of each floor varies. Most floors have an area over 2000 m^2 and much of the external façade is glazed, thus this building lies outside the range of applicability of the traditional design methods.

The 9^{th} floor was selected by the structural fire engineers on the project as the most onerous floor from a structural perspective, as this floor has the longest beam spans as well as slender diagrid members compared to those found lower in the building. This is where a severe fire is most challenging for the structure and therefore the first location to be studied. It is anticipated that as the methodology becomes more mature, the assessment of the most onerous floor, or floors, will also be made from a fire perspective and the assessment done on both the most onerous fire floor (worst case fire) and most onerous structural floor (worst case structural floor).

2.2 Input Parameters

The specific dimensions of the 9^{th} floor of the Mumbai C70 building have been applied to the methodology presented. The total floor area, A_t, is 2846 m^2 and the floor to ceiling height, H, is 3 m.

The fuel load density, q_f, is taken as 570 MJ/m2, as per the 80^{th} percentile design value [15] for office buildings. The heat release rate per unit area, $\dot{Q}''$, is taken as 500 kW/m^2 which is deemed to be a typical value for densely furnished spaces, as design guidance [16] gives this value for retail spaces. Based on these two values, the characteristic burning time, t_b, is calculated by *Eq. (2)* to be 19 minutes.

3 RESULTS

Applying this methodology to a family of fires results in a range of far field temperatures and total burning durations, as shown in *Table 1*. These results illustrate the concept that hotter far field temperatures will occur for larger fires, but for shorter durations.

The far field temperatures are plotted in *Fig. 5* along with the temperature-time curves of traditional design methods. The growth and decay phases of the gas temperatures for the curves from this methodology are assumed to be very fast [3]. This is because the larger an enclosure is, the lower the importance of the thermal inertial of its linings are, thus the faster the growth and decay phases will be, i.e. the transport of the hot gases in the smoke layer is faster than the heat transfer to the surfaces. Note that the cooling is not neglected in the structural analysis, only the decay phase is eliminated from the fire environment.

Percentage of Floor Area	Burning Area A_b (m^2)	Total Heat Release Rate $\dot{Q}$ (MW)	Total Burning Duration t_b (min)	Far Field Temperature T_{ff} (°C)
1%	28	14	1900	156
2.5%	71	36	760	258
5%	142	71	380	376
10%	285	142	190	543
25%	711	356	76	803
50%	1423	711	38	1118
100%	2846	1423	19	*1200 (near field only)*

Table 1. Results for the family of travelling fires

The standard temperature-time curve extends to a region of temperatures and burning times that for the present building cannot be explained in terms of the possible fire dynamics. Two Eurocode 1 Parametric temperature-time curves are given to represent different amounts of assumed glass breakage; 25% glass breakage to produce a longer, relatively cool fire and 100% glass breakage to produce a shorter, relatively hot fire. Note that *Fig. 5* provides the far field values that need to be combined with the near field to produce curves of the type shown in *Fig. 3*. Every point on the floor plate will at some point experience the near field temperature for the burning time (19 minutes) and the far field temperature for the rest of the total burning duration, as illustrated in *Fig. 1*.

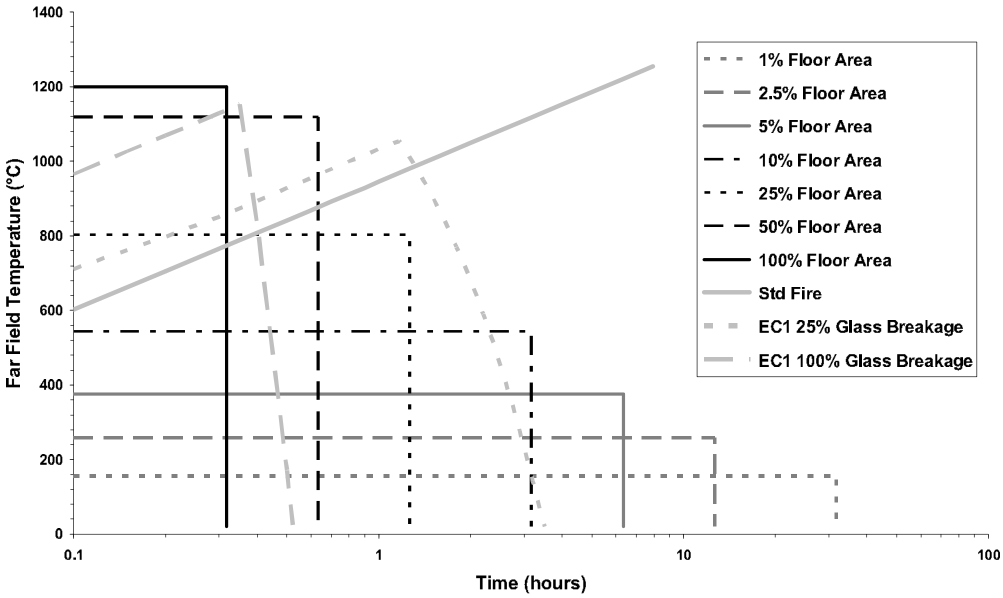

Fig. 5. Far Field Temperatures for the Family of Fires and Nominal Design Methods

The next step of this methodology is to apply these curves to a structural fire model. The fire engineer and structural engineer should work together to identify which curves are the most challenging to the structure. It is anticipated that the smaller area fires (1% through 5% of the floor area) would be less challenging, while the larger area fires (10% and above) would be more challenging, especially the cases with longer total burning durations.

4 CONCLUSIONS

This work develops a novel methodology to determine design fires for structural fire analysis of modern buildings that are outside the range of applicability of traditional methods. It has been applied to determine a family of fires for a floor in the Mumbai C70 building. The family of fires was generated by considering different burning areas that travelled on the floor plate. The methodology calculated both the near and far field temperatures to characterise the thermal environment for structural analysis. Future work will develop the tool used to determine the temperature fields, examine the effect of the path of fire travel, as well as determine the impact of this family of fires on the structure.

5 ACKNOWLEDGMENTS

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EVALUATION OF THE FIRE RESISTANCE OF A SPORT HALL USING STRUCTURAL FIRE ENGINEERING

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INTRODUCTION

It is the purpose of this paper to present a study on the needs of passive fire protection in the steel roof structure of the Oporto Football Club (FCP) sport hall, to fulfil a fire resistance of 60.

Due to the large dimension of the sport hall a prescriptive approach using the standard fire curve ISO 834 revealed to be too severe, quite unrealistic and uneconomical. Indeed the fire load of this type of construction is generally rather small being not possible to reach the high temperatures of the ISO curve. On the other hand a large amount of air is available, which is a second factor for reducing the temperatures in a real fire [1]. The results obtained with the ISO fire curve were compared with the ones obtained using the natural fire, in accordance with the advanced calculation methods included in the recently approved part 1-2 of EC1 [2]. The program Ozone V2.2 [3,4] was used to simulate the natural fire, and the thermo-mechanic behaviour of the structure was modelled by the finite elements program SAFIR [5].

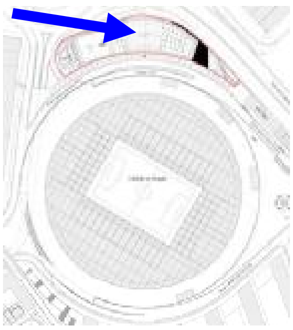

Fig. 1. FCP sport hall plant

Fig. 2. FCP sport hall

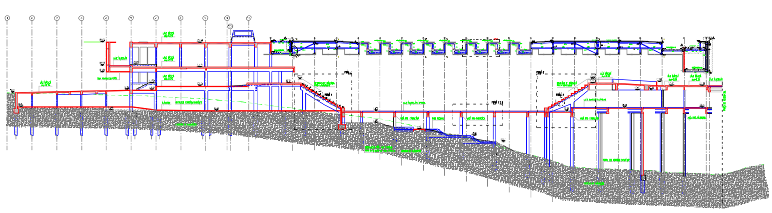

Fig. 3. Inside draw of the FCP sport hall

1 FIRE SCENARIOS

From the point of view of the characterization and prediction of the fire development, it was studied an equivalent compartment to the pitch zone and the main entrance, with the same air

volume, being considered localized fire and fire in whole pitch zone, in a total of seven fire scenarios.
Although Eurocode 1 allows for the consideration of the beneficial effect provided by active safety measures against fires, in this study these measures were not considered, according the Portuguese National Annex.
In the natural fires studied was considered a value for the maximum Rate of Heat Realese (RHR) of 250kW/m^2, a fire load density of 200MJ/m^2 and a medium fire grow rate ($t_\alpha = 300$ s) or a fast fire grow rate ($t_\alpha = 150$ s) depending on the situation [1].

1.1 Fires in the entire pitch

The pitch has an area of 26.2x48=1257.6m^2 (being equal to the fire area). The height of the compartment is 13.5m. The area of openings is 97.4m^2 located at a height of 6.6m. The maximum fire area was 2262.5m^2.

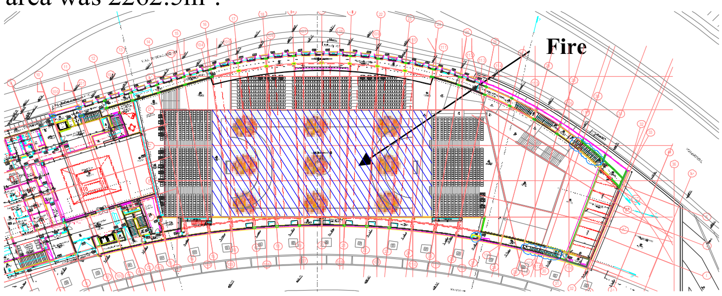

Fig. 4. Fire in the entire pitch

The first three scenarios are related to the existence of fire in the entire pitch.
Scenario 0 - standard fire ISO834.
Scenario 1 - natural fire with a linear openings variation from 10% at 20ºC until 50% at temperature of 400ºC and after until 100% openings at 500ºC and a medium fire grow rate.
Scenario 2 - natural fire with constant opening area during the fire. Several constant percentage of openings were tested (from 10% until 100%), resulting on the use of the most severe case (60% openings) and a medium fire grow rate.
The graphics in figures 5 and 6 show the temperature evolution in the compartment for the two last fire scenarios, obtained with the program Ozone [3, 4].

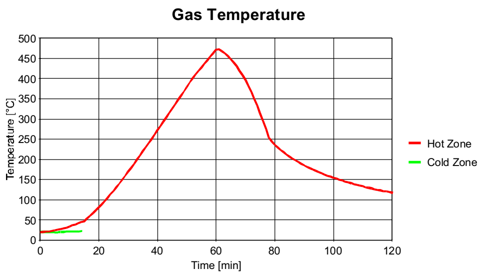

Fig. 5. Temperature in scenario 1

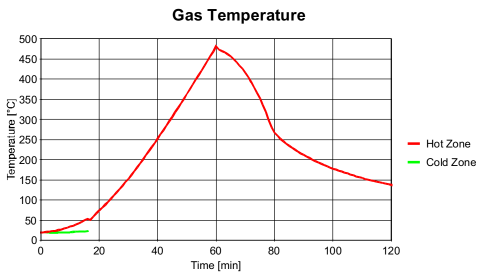

Fig. 6. Temperature in scenario 2

1.2 Localized fire in the pitch

Scenario 3 - The fire area was 9m^2, corresponding to a diameter of 3.4m [4], at the pitch level (Figure 7). The opening variation was equal to the one on scenario 1, however due to the large amount of oxygen it was found that the fire is controlled by the fire load not depending on the openings. Again, the maximum fire area was $2262.5m^2$ and a medium fire grow rate.

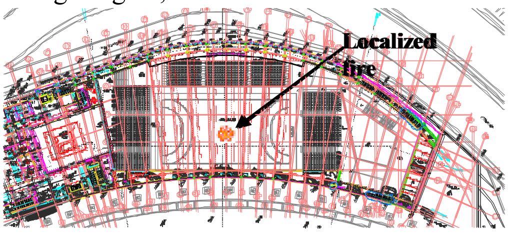

Fig. 7. Localized fire in the pitch

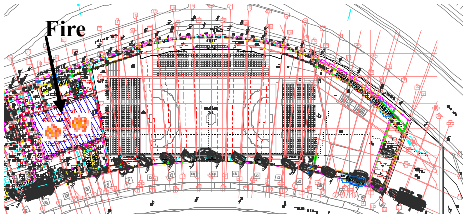

Fig. 8. Fire in the vip foyer

Figure 9 show the average temperature evolution in the compartment for this fire scenario.

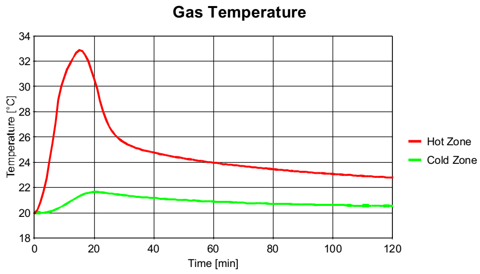

Fig. 9. Temperature in scenario 3

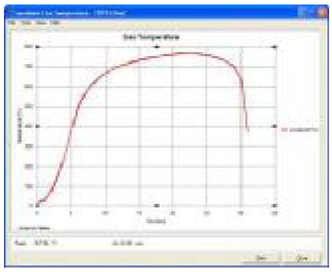

Fig. 10. Temperature in scenario 4

In this case the fire remained localized, and the flame did not impact the ceiling. With the Heskestad model [2, 6] it was possible to determine the maximum temperature at the covering with the value of 60ºC, slightly higher than the one obtained in OZone V2.2 for the average temperature in the compartment (see Figure 9).

1.3 Localized fire in the Media balcony

Scenario 4 - The fire area was $36m^2$, corresponding to a diameter of 6.8m [4], at media balcony (10.2m from the pitch level). The opening variation was the same used on scenario 1, however due to the large amount of oxygen it was found that the fire is controlled by the fire load not depending on the openings. The maximum fire area was $2262.5m^2$ and a fast fire grow rate.

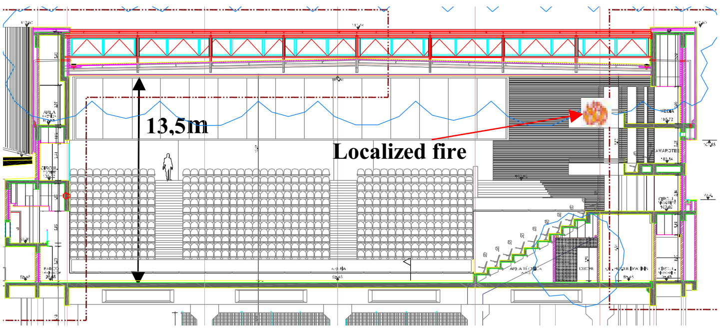

Fig. 11. Localized fire in the media balcony

In this case the fire remained localized, and the flame was impacting the ceiling. With the Hasemi model [2] and a RHR of $500kW/m^2$ it was possible to determine the maximum temperature at the lower cord level (767.96 ºC), which was significantly higher than the average temperature obtained with OZone V2.2. Figure 10 show the temperature evolution in the compartment for this localized fire.

1.4 Fires in the vip foyer

The vip foyer has an area of 22x15=330m^2 (being equal to the fire area and to maximum fire area). The height of the compartment is 3.0m. The openings area is 45m^2, corresponded to the façade turned into the pitch (see Figure 8).
Two scenarios were considered:
Scenario 5 - natural fire with a linear openings variation from 10% at 20ºC until 50% at temperature of 400ºC and after until 100% openings at 500ºC.
Scenario 6 - natural fire with constant opening area during the fire. Several constant percentage of openings were tested (from 10% until 100%), resulting on the use of the most severe case (100% openings)
The graphics in figures 12 and 13 show the temperature evolution in the compartment for this two fire scenarios where a medium fire grow rate was considered.

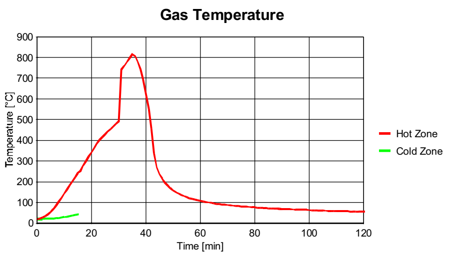

Fig. 12. Temperature in scenario 5

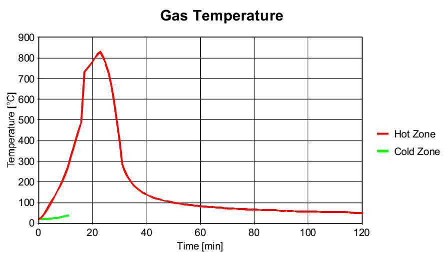

Fig. 13. Temperature in scenario 6

2 MECHANICAL ANALYSIS

In the safety evaluation of the steel structure were used: simplified methods prescribed in part 1.2 of EC3 [7] and implemented in the software Elefir-EN [8] (developed at Universities of Aveiro and Liege); and advanced calculation model using the software SAFIR [3] (developed at University of Liège).
The structural system analysed was a truss in steel S275, with a span with 36m length. It was chosen to evaluate only the truss subjected to the most severe conditions. The pofiles used in the truss were: HEA 120, HEA 140, HEA 160, HEA 240, HEA 260, HEA 280 and HEB 240.

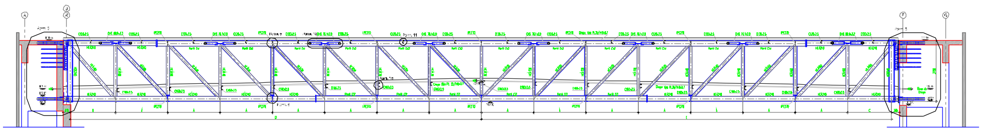

Fig. 14. Truss subjected to the most severe conditions

2.1 Mechanical actions

EN 1990 [9] defines fire as an accidental situation and according to the Portuguese National Annex the following load combination was used:

$$\sum G_k + \psi_{1,1} \cdot Q_{k,1} + \sum \psi_{2,i} \cdot Q_{k,i} + \sum A_d \qquad (1)$$

According to the Annex A1 of EN1990 the combination factores ψ_1 and ψ_2 take the value of 0.0 for variable imposed loads on roofs and the accidental combination of actions results in

$$1.0G_k + \psi_{1,1}Q_{k,1} = 1.0G_k + 0.0Q_{k,1} = G_k \qquad (2)$$

2.2 Simplified methods

For this accidental combination, the most severe efforts for the several profiles are presented in table 1. In this table it is found also the critical temperatures obtained with Elefir-EN and the maximum temperatures in the steel profiles obtained with Ozone V2.2.

Table 1. Most severe efforts at 20ºC, critical temperatures corresponded to the fire scenarios

Profiles	Efforts [kN] Compr.	Efforts [kN] Tension	Critical Temp. (Elefir-EN) [º C]	$T_{máx}$ scenario 2 (OZone) [º C]	$T_{máx}$ scenario 6 (OZone) [º C]	Critical time ISO834 [min]	Temp. after 60min of ISO834 [ºC]
HEA280	-110.3		943	435	735	62	938
HEA260		144.1	899	438	736	47	938
HEA240	-212.5		774	440	738	29	939
HEB240		3.74	1195	404	781	320	954
HEA160	-82	113	808	451	753	27	940
HEA140	-60	89	814	455	765	26	941
HEA120	-40	56	859	457	769	26	941

Table 1 shows that the maximum temperatures never reach the critical temperatures with natural fire. In here it is shown the results for the fire scenarios 2 and 6. Regarding the fire scenario 4 the maximum temperature of the compartment at the level of the lower cord of the truss is 767.96 ºC (Fig. 10), meaning that again the critical temperatures are never reached (as an example it should be mentioned that for the profile with the biggest section factor, HEB 240, the maximum temperature obtained in this scenario is 707 ºC). It can be concluded that there is no need to use passive protection against fire, which would not be true if the standard fire curve ISO834 was used.

2.3 Advanced calculation methods

The most loaded truss was analysed (Figure 14) with the load combination (2) and the fire scenario 2 or the ISO834 fire curve in all the 4 sides of the profiles. With these fire scenarios it was first performed thermal analysis to each profile, using the program SAFIR (figure 15).

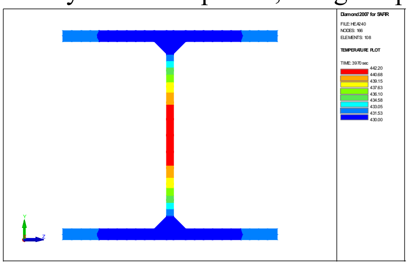

Fig. 15. Maximum temperatures in the HEB240

Following the thermal analysis, it was made a 2D structural analysis, using a material and geometrical non-linear analysis, with beam finite elements.
The analyses performed were:

1. Truss without the possibility of expanding longitudinally subjected to ISO834;
2. Truss with the possibility of expanding longitudinally subjected to ISO834;
3. Truss without the possibility of expanding longitudinally subjected to scenario 2;
4. Truss with the possibility of expanding longitudinally subjected to scenario 2;

Figures 16 and 17 show the deformed shapes immediately before collapse for analysed cases 3 and 4.

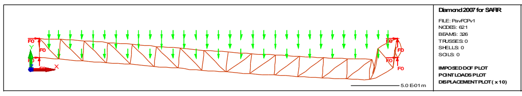

Fig. 16. Deformed shape for case 3

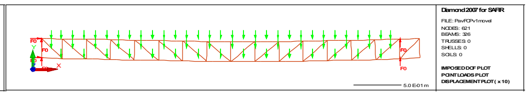

Fig. 17. Deformed shape for case 4

Table 2 shows the time needed for the collapse, the maximum horizontal reaction in the supports and the maximum horizontal displacements in the supports.

Table 2. Final results

Case study	Collapse time [min]	Reaction [kN]	Displacement [cm]
1) ISO 834	24.8	2100	-
2) ISO 834	26.6	-	43
2)* ISO 834	60	2100	-
3) scenario 2	66.1	2100	-
4) scenario 2	No collapse	-	21

2)* truss protected to hold 60 min of ISO834 fire curve

3 CONCLUSIONS

It was shown that the temperatures at the level of the roof are relatively low and no passive protection against fire in the steel structure was needed, as a prescriptive evaluation using the standard ISO834 fire curve would impose. Adopting the natural fire, was proven that the steel profiles of the truss have a fire resistance higher than the 60 minutes, being even able to not collapse during the entire fire development, if the truss can freely expand in the supports. This can be obtained from the structure configuration or possible yielding of the supports for the horizontal reactions of 2100kN. However this possibility does not require to be used, due to the fact that the structure holds without collapse 66 minutes, enough time for the evacuation of the occupants and for fire brigades intervention.

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DESIGN OF COLD-FORMED STAINLESS STEEL TUBULAR COLUMNS AT ELEVATED TEMPERATURES

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INTRODUCTION

In the past 15 years, there has been a growing interest in the use of stainless steel in structural applications owing to the high strength, attractive appearance, corrosion resistance and fire resistance of the stainless steel material [1]. Recently, significant progress has been made on the experimental investigation of the structural behaviour of stainless steel columns which enables improvement of the existing design specifications [2]. Comprehensive design guidance for the design stainless steel structural members at ambient temperature is given in the European Code [3], American Specification [4] and Australian/New Zealand Standard [5]. The fire resistance of stainless steel, however, receives relatively little attention with only a few design guidance available, such as the European Code and design guidance proposed by Gardner and Baddoo [6] and Ng and Gardner [7]. These design guidance were developed to determine the failure temperature of a stainless steel structural member subject to a constant load. The performance of cold-formed steel structural members at elevated temperatures has been investigated by Ranby [8], Feng et al. [9], Dharma and Tan [10], Tan et al. [11], Lim and Young [12] and Chen and Young [13], but the works have been focused on carbon steel rather than stainless steel structural members. In order to improve our understanding on fire resistance of stainless steel members, the structural performance of stainless steel members at elevated temperatures has to be investigated.

While physical tests are time consuming and expensive to conduct, numerical analyses can be carried out efficiently and with sufficient accuracy in terms of both time and cost. To and Young [14] investigated the performance of concentrically loaded cold-formed stainless steel tubular columns at elevated temperatures using FEA and performed a parametric study to study the effects of cross-section geometries and temperature on the strength and behaviour of the columns. Based on the numerical results obtained, two design rules for cold-formed stainless steel tubular columns at elevated temperatures have been proposed and presented in this paper. Furthermore, reliability analysis was also performed to assess the validity of the proposed design rules.

1 FINITE ELEMENT ANALYSIS

To and Young [14] developed a finite element model (FEM) to assess the structural performance of cold-formed stainless steel tubular columns at elevated temperatures using the finite element package ABAQUS. The model was developed to simulate fixed-ended columns. The two ends of the column were fixed against all degrees of freedom except that the column was allowed to displace along the loading direction at the loaded end, and all other nodes in the column were free to translate and rotate in any directions. The four-noded doubly curved shell element S4R with six degrees of freedom per node was used in the model. A mesh size of approximately 20 mm$\times$10 mm (length by width) was chosen for the flat portions of the columns and finer mesh was used at the corners. Initial imperfections and material non-linearity described by the stress-strain model proposed by Chen and Young [15] were incorporated in the model. The FEM involves two types of analyses: (1) the Eigenvalue analysis to determine the buckling modes of the column and (2) the load-displacement non-linear analysis to determine the ultimate loads, failure modes and axial shortenings of the columns. The model has been verified against column test results obtained by Young and Lui [16]. Both the failure loads and failure modes of the columns have been accurately predicted by the FEM. The mean value of the tested-to-FEA load ratio is 1.00 and the corresponding coefficient of variation (COV) is 0.065 in the verification. The good agreement of the results has demonstrated the validity of the FEM.

A parametric study has been carried out by To and Young [14] using the verified FEM to study the effects of cross section geometries, plate thickness, effective column length and temperature on the strengths and failure modes of the columns. Three series of rectangular hollow sections (Depth$\times$Width): R1 (120mm$\times$60mm), R2 (80mm$\times$50mm) and R3 (140mm$\times$80mm) and one series of square hollow section S1 (50mm$\times$50mm) were investigated in the parametric study. The series have different thicknesses ranging from 1mm to 6mm and different column lengths ranging from 300mm to 3600mm. The temperatures ranged from 22°C to 900°C were investigated. A total of 327 columns were analysed in the study. The developed finite element model and the parametric study are detailed in To and Young [14].

2 DESIGN RULES

Based on the numerical results obtained from the parametric study conducted by To and Young [14], two different methods to determine the failure loads of cold-formed stainless steel tubular columns at elevated temperatures were proposed. First, the column strength equations in Eurocode 3 Part 1.4 [3] were employed with the substitution of reduced material properties at elevated temperatures into the equations. Second, the column design rules were developed based on the ultimate strength of stainless steel material and gross sectional area of the columns rather than the yield strength (0.2% proof stress) and effective area.

2.1 Column strength design rules in Eurocode 3 Part 1.4

The column strengths obtained by To and Young [14] were compared with the design strengths calculated from Eurocode 3 Part 1.4 [3] but with the replacement of ambient temperature mechanical properties by the elevated temperatures mechanical properties. It is shown that the column strengths agree fairly well with the strengths obtained from FEA. As shown in *Fig. 1*, the mean value of the FEA-to-design load ratio R_{FEA} / R_{EC3} is 1.27 and the COV is 0.12, where R is ratio of the failure stress f_f of the column (failure load divided by gross sectional area) obtained from either the EC3 equations or FEA to the ultimate strength of stainless steel $f_{U,T}$ at temperature T°C.

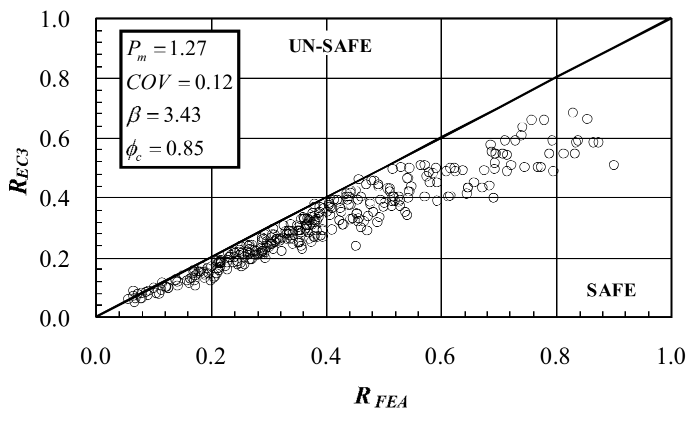

Fig. 1. A plot of R_{EC3} against R_{FEA}

In order to ensure a reliable design rule, reliability analysis was performed, in which the reliability is measured by the reliability index β. The NAS Specification [17] for cold-formed steel structures recommended a minimum reliability index of 2.5 for structural members, while the ASCE Specification [4] for cold-formed stainless steel structures recommended a minimum reliability index of 3.0. In this study, a minimum reliability index of 2.5 was adopted. The resistance factor ϕ_c of 0.85 was used in the analysis for concentrically loaded compression members, which is given by the NAS Specification [17] and ASCE Specification [4]. A load combination of 1.2DL + 1.6LL as specified in the American Society of Civil Engineers Standard [18] was used in the analysis, where DL is the dead load and LL is the live load. The dead load-to-live load ratio is 0.2. The statistical parameters M_m = 1.10, F_m = 1.00, V_M = 0.10, and V_F = 0.05, which are the mean values and COVs for material properties and fabrication factors were obtained from Table F1 of the NAS Specification [17]. The statistical parameters P_m and V_P (COV) are the mean value and coefficient of variation of FEA-to-predicted load ratio respectively, as shown in *Fig. 1*. A correction factor C_P was used to account for the influence of a small number of tests, and a factor C_P is given in Eq. (F1.1-3) of the NAS Specification [17]. The equation to obtain β is shown in *Eq. (1)* below:

$$\beta = \frac{\ln\left(\dfrac{M_m F_m P_m}{0.657\phi_c}\right)}{\sqrt{\left(V_M^2 + V_F^2 + C_P V_P^2 + 0.21^2\right)}} \geq 2.5 \tag{1}$$

The reliability index was calculated to be 3.43, which is greater than the minimum required value of 2.5. Therefore, the column strengths calculated from the buckling resistance equations in the Eurocode 3 Part 1.4 [3] using the reduced material properties at elevated temperatures is reliable. It can be seen from the plot of R_{EC3} against R_{FEA} in *Fig. 1* that the design equations of Eurocode 3 are generally conservative as most of the data points lie below the 45° straight line. Hence, it is suggested that this design rules could be used for determining the buckling resistance of cold-formed stainless steel tubular columns at elevated temperatures using reduced material properties.

2.2 Column strength design rules based on ultimate strength of material and gross sectional area

In the existing design specifications, the calculation of column strength of cold-formed stainless steel structural members is always based on the yield strength (0.2% proof stress) of the material. However, this philosophy may not be appropriate as it can be seen in the cold-formed stainless steel square hollow section column tests conducted by Young and Lui [16] and Liu and Young [19] that the failure stress for some of the short columns, in which the failure mode is yielding, was close to the ultimate strength of the stainless steel material instead of its yield strength. The existing design specifications might therefore underestimate the strengths of the short columns. In addition, the determination of strength of thin-walled sections is normally based on the concept of effective area A_e. Its calculation, however, could be somewhat tedious. In view of these, this study attempts to propose design rules for stainless steel columns based on ultimate strength of the stainless steel material and the gross sectional area, in which the concept is similar to the direct strength method [20, 21]. To start with, the failure load of the column P is related to the gross area of the section A_g with the failure stress f_f as shown in *Eq. (2)*:

$$P = A_g f_f \tag{2}$$

Analytically, the failure stress could be written as a function of the ultimate strength of stainless steel material $f_{U,T}$, the elastic flexural buckling stress $f_{F,T}$, and the elastic local buckling stress $f_{L,T}$ of the column at temperature T°C given by:

$$f_f = F(f_{U,T}, f_{F,T}, f_{L,T}) \tag{3}$$

Normalizing both sides by the ultimate strength $f_{U,T}$ of the stainless steel material at temperature T°C, *Eq. (3)* becomes:

$$R_U = \frac{f_f}{f_{U,T}} = F\left(\frac{f_{F,T}}{f_{U,T}}, \frac{f_{L,T}}{f_{U,T}}\right) \tag{4}$$

In *Eq. (4)*, the temperature effect on the strength of the column would not be obvious as the normalization of ultimate strength is temperature dependent as well as the elastic flexural buckling stress and elastic local buckling stress are also temperature dependent. The temperature effect could be considered by including the elastic modulus at a temperature normalized with the elastic modulus at room temperature E_{22}. Hence, *Eq. (4)* can be modified as:

$$R_U = F\left(\frac{f_{F,T}}{f_{U,T}}, \frac{f_{L,T}}{f_{U,T}}, \frac{E_T}{E_{22}}\right) \tag{5}$$

Now, postulate that the load ratio R_U can be expressed as a function of a dimensionless slenderness ratio λ given by:

$$R_U = F\left[\lambda = \left(\frac{f_{F,T}}{f_{U,T}}\right)^a \left(\frac{f_{L,T}}{f_{U,T}}\right)^b \left(\frac{E_T}{E_{22}}\right)^c\right] \tag{6}$$

where a, b and c are constants. Appropriate values of a, b and c have to be determined such that the R_U-λ relation can be represented by a simple mathematical function. By trial and error, it was proposed that a = -0.25, b = -0.25 and c = -0.45 and hence *Eq. (6)* can be re-written as:

$$R_U = F\left[\lambda = \left(\frac{f_{U,T}^2}{f_{F,T} f_{L,T}}\right)^{0.25} \left(\frac{E_{22}}{E_T}\right)^{0.45}\right] \tag{7}$$

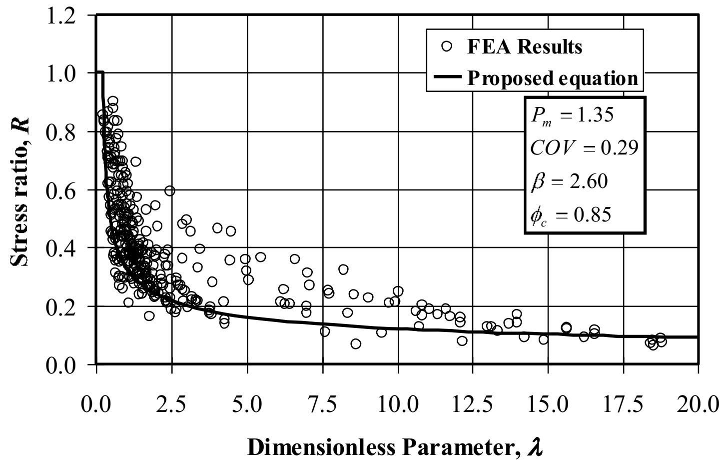

Fig. 2. Comparison of FEA results with the proposed equation

Fig. 2 shows the plot of R_U against λ, in which a decreasing trend of R_U with increasing value of λ was observed. The next step is to devise the function F to describe the R_U-λ relationship. Subjected to the constraint that the reliability index must be greater than or equal to 2.5, and that the mean value of the FEA-to-predicted load ratio is to be as close to 1.0 as possible, an iterative curve fitting process was carried out. Finally, the following equations are proposed:

$$R_U = 1.0 \qquad \text{for } \lambda \le 0.24 \tag{8}$$

$$R_U = 0.34\left(\frac{\lambda}{0.75} - 0.25\right)^{-0.4} \qquad \text{for } \lambda > 0.24 \tag{9}$$

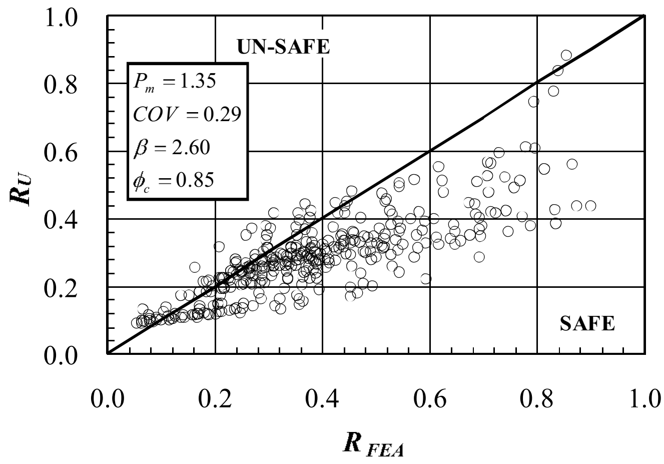

Fig. 3. A plot of R_U against R_{FEA}

Fig. 2 compares the FEA results with the prediction by the proposed equations. Reliability analysis was carried out and the results give $\beta = 2.60$, $P_m = 1.35$ and COV = 0.29. A plot of R_U against R_{FEA} is also shown in *Fig. 3*. Hence, it can be concluded that the proposed equations are reliable and slightly conservative. Finally, the failure load of a column can be computed from:

$$P = R_U f_{U,T} A_g \tag{10}$$

The derivation of the proposed design equations and the comparison of the predicted column strengths with test results are detailed in To and Young [14].

3 CONCLUSIONS

Design rules of cold-formed stainless steel tubular columns at elevated temperatures have been proposed. Two design rules based on different design philosophies have been studied. The first design rules is simply the substitution of the mechanical properties of stainless steel material at elevated temperatures into the buckling resistance equations of columns at room temperature specified in the European code for stainless steel structures. The second design rules was developed based on the ultimate strength of the material and the gross sectional area of the column, in which the failure stress of the column is said to be a function of the elastic local buckling stress, elastic flexural buckling stress and the elastic modulus at the temperature of concern. The calculated column strengths were compared with the finite element analysis results of 327 columns, and the reliability analysis showed that both design rules are generally reliable and conservative. Therefore, it can be concluded that the two design rules can be safely employed for estimating the column strengths of concentrically loaded cold-formed stainless steel tubular columns at elevated temperatures.

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APPLICATION OF STRUCTURAL FIRE ENGINEERING TO THE STEELWORK DESIGN OF CANNON PLACE, LONDON

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INTRODUCTION

Cannon Place is a new office building that is currently being built above Cannon Street Station in Central London, with train lines running below the building. The building is over 30m in height and is served by four firefighting cores located around the perimeter of the building. There are two atria that run the height of the building and are linked at the lowest level of the atria.

The project involves the design of the new office building as well as extensive modifications to the station below. The architectural and structural design of the office building is being carried out by Foggo Associates. Bodycote warringtonfire are designing the fire safety strategy for the project.

The structural design uses a highly innovative approach, incorporating an extensive external framed structure that is cantilevered to the front and rear of the building. As the structure is external, the visible appearance of the structural elements is crucial to the architectural concept for the building.

Bodycote warringtonfire have carried out detailed fire engineering analysis on the entire structure of the building based on the approaches in PD 7974 and the Eurocodes. The analysis included an assessment of the impact of fire on the internal structure as well as flame projection and impact on the external structure, resulting in recommendations on the amount of fire protection required to ensure that the structure will not fail in the event of a fire.

1 OVERVIEW OF ANALYSIS

To comply with Approved Document B[1] (ADB), for office buildings more than 30m above ground level the building need to be sprinklered throughout and the elements of structural frame need to be constructed to achieve a 120 minute standard of fire resistance when tested to BS 476[2]. The aim of this investigation is to show that the requirements of the UK Building Regulations 2000[3] can be met with a reduced level of fire protection using structural fire engineering analyses.

Requirement B3 of the Building Regulations states that "The building shall be designed and constructed so that, in the event of a fire, its stability will be maintained for a reasonable period." The intention is to go beyond that requirement and to demonstrate that, in the event of a foreseeable fire, the building will suffer no structural collapse throughout the entire duration of the fire. The reduction in the amount of fire protection in certain areas of the building will therefore not reduce the level of safety or reduce the performance of the building in a fire. The reduction in fire protection is simply to reduce or eliminate fire protection for parts of the structure where it performs no useful purpose. Any structure that only supports a roof would not need any fire protection (as recommended in ADB) and so has not been analysed.

Different elements of structure will be subject to different fire scenarios depending on their location and characteristics and as such, the assumptions taken for the analysis would differ from one element to another. It should also be noted that the deck structure and any element of structure penetrating the deck or going through the concourse would be protected to a 120 minute standard to maintain an adequate level of separation between the station and office building.

2 ANALYSIS METHODS

The intention is to carry out an analysis of the 'reasonable worst case' fire in each individual area to determine its impact on the structure. The sprinkler intervention has also been ignored, in respect of its actions upon the fire, however the provision of sprinklers allows the selection of a less conservative fire load and thus the provision is taken into account in an indirect manner. The fire is assumed to burn until it runs out of fuel. Fire brigade intervention is ignored, which is a highly onerous assumption as the fire service would be likely to either extinguish or, at least significantly reduce the severity of any fire.

The procedure of the analysis is:

a) Analyse the potential fires in each area of the building that are to be reviewed;
b) From a), determine the temperature and duration of flames that may impact on any structure in the vicinity;
c) Analyse the impact of the flames on the structure (i.e. the temperature that the steel may be heated to);
d) Determine whether the maximum calculated steel temperatures would lead to structural failure, and if so, what level of fire protection would be required when tested to BS 476.

2.1 Potential Fire Sizes

One of the factors that needs to be assessed in the analysis is the potential size and duration of any fire that might occur. In order to predict this it is necessary to understand how fires develop.

One of the most important factors for this is to determine the risk of a flashover occurring. Flashover is a phenomenon that occurs when the smoke layer above a localised fire in one part of a compartment reaches about 600°C. At this point the level of radiation from the smoke layer is sufficient to cause spontaneous ignition of all the combustible surfaces within that room. This causes a very rapid transition from a localised fire in one part of the room to a fully developed fire that involves the entire space.

If a fire occurs within an enclosed space, there is a risk of flashover occurring. However, if there is no possibility of a hot smoke layer developing, then flashover cannot occur. This is the situation in cases where the ceiling is well above the fire or where there is a large amount of permanent ventilation. In these scenarios the potential fire size would be considerably smaller as the only method of fire spread would be via radiation from the flames, which is a much slower process.

It is therefore necessary to determine the likelihood of a flashover fire occurring in each area and then to determine the potential fire size.

2.2 Impact of Fires on Structural Steel

Steel gradually reduces in strength as it is heated, so the structure of a building would stay in place until the steel is heated to the point at which the remaining strength is insufficient to sustain the load imposed on it. At this point the steel would begin to sag. Tests carried out by BRE at Cardington showed that when this happens, the load that is supported by that member tends to become redistributed to other elements within the structure, so that full failure is unlikely to occur.

This analysis is intended to ensure that any structural element will not be heated to the point at which it may start to fail and will always have sufficient strength to support the imposed load, so that even the initial sagging will not occur.

Steel has approximately half its strength remaining when it reaches 550°C. General structural design codes tend to end up with designs that have a factor of safety of at least 2. This is in addition to the fact that the structural design is generally based on the highest loading that will ever occur during the lifetime of the building. During normal conditions, the structure is loaded to a relatively low level. It can therefore be concluded that as long as the steel is kept below approximately 550°C, it will not fail. This corresponds with the guidance of BS 5950: Part 8[4].

Even steel that is unprotected has a certain degree of fire resistance that depends on its section size. The larger the steel section, the longer it will take to heat up in a fire. Providing fire protection to a steel element slows the rate at which it heats up, effectively increasing its fire resistance.

3 FIRE SCENARIOS

The building will be used as an office building. Due to the height of the building, every floor would be constructed as a 120 minute compartment floor. There are two fully glazed atria of equal size that run the full height of the office levels and are joined at the base by a large reception area. A temperature control smoke extraction system is provided to the atria as such the glazing enclosing the atria will only be smoke retarding but not fire rated. The building would also be protected by an automatic sprinkler system. Therefore, it is reasonable to assume that a fire would be limited to the area of operation of the few sprinkler heads directly in the vicinity of the fire and would not involve a complete compartment (a whole floor in this case). However, in order to add a margin of safety to the results, it is proposed to assume sprinkler failure and the fire involving the compartment of origin, as detailed below. This will allow additional confidence in the results.

Potential tenancies may subdivide the internal layout in a variety of ways. This sub-division can affect the temperature and duration of the fire and in order to take this into account a number of fire scenarios were assumed and then modelled:

a) Fire involving the whole main floor plate;
b) Fire affecting a single tenancy (approx 1/3 of the floor plate);
c) Fire affecting an area approx 1/3 of a tenancy;
d) Fire affecting an office layout, area approx $100m^2$;
e) Fire affecting an office layout as designed by Foggo architects, being equivalent to 2/3 of a tenancy.

The scenarios of the fires were assumed to be post-flashover fires within that space. The amount of ventilation that is available to the fire was included in the analysis. This would be dependent on how much of the enclosure of the room was open at the start of the fire (e.g. an open door or window) and on the failure of any additional sections of the enclosure during the fire. Glazing will often stay in place during the early stages of a fire, but once flashover occurs the temperatures within the room tend to be sufficient to cause most, if not all, of the glass to fail.

The external wall to the building on all levels is glazed. Sensitivity studies were therefore carried out on the amount of ventilation that might be available. The maximum limit would be to assume all the glass failed. The minimum assumed was that only 75% of this failed. Further sensitivity studies involved examining different fire loads and wall linings.

The fire load was taken to be the 80% fractile for office accommodation ($570MJ/m^2$) as given in PD 7974: Part 1, and then examined with the following safety factors specified in PD 7974: Part 3:

$\gamma_1 = 2.2; \qquad \gamma_2 = 1.2; \qquad \gamma_3 = 0.6$

These safety factors relate to the height, use and occupancy of the building and are used to calculate the final fire loads that are used in the analysis.

The structures that could be affected by these types of fire are:

a) The external façade bracing;
b) The internal frame columns and beams above deck level;
c) The floor slabs above deck level.

The deck structure and the structure below deck level will not be affected by these office fires.

4 INTERNAL STRUCTURE

4.1 Details of Analysis

The initial stage of the analysis of a fire within the internal floor areas was to model the potential severity of a fire within this space and compare this with an equivalent period within the standard BS 476: Part 20 fire resistance furnace test. This approach is called the Time Equivalent method. Two different methods were used to achieve this: a) Direct Method; b) Graphical Method. The use of two independent methods gives additional confidence in the results of the analysis.

The Direct Method consists of an equation, which was derived from experimental data that directly compares the compartment data (i.e. compartment size, ventilation area, amount of fire load etc.) to an equivalent duration within the standard fire resistance furnace test. This is a relatively simple approach that does not give specific data on the temperatures reached within the compartment, but gives an easily understood result. Equation 31 from PD 7974: Part 3 was used.

The Graphical Method is a more detailed calculation that consists of a number of stages:

a) Predict the time/temperature curve that would occur in the event of a fire within the compartment (based on input data such as compartment geometry, ventilation area and fire load). The Parametric Fire Curve from Eurocode 1-1-2[5] was used for this stage.
b) Analyse the effect of the fire on a typical protected steel section. Modify the thickness of the fire protection until the peak temperature of the steel reaches 550°C (other limiting temperatures could also be used). Equation 66 from PD 7974: Part 3 was used for this stage.
c) Analyse the same steel element with the same thickness of fire protection when exposed to BS 476 and calculate the time at which the steel reaches the same maximum temperature as was reached in the predicted fire. This would give the equivalent severity of the predicted fire in terms of duration within the BS 476 test.

4.2 Results

The analysis was carried out using an in-house computer program. In total 24 different scenarios have been investigated. The detailed results of the analysis are presented in Table 1.

Table 1. Description of Fire Scenarios and Results

Scenario		Direct Method	Graphical Method
1	Whole floor plate, level 2, BASE	52 mins	54 mins
2	One tenancy, level 2, BASE	51 mins	49 mins
3	1/3 of a tenancy, level 2, BASE	51 mins	47 mins
4	Office floor plate, level 2, BASE	51 mins	36 mins
5	Whole floor plate, level 2, 75% glazing failure	63 mins	67 mins
6	One tenancy, level 2, 75% glazing failure	54 mins	57 mins
7	1/3 of a tenancy, level 2, 75% glazing failure	53 mins	55 mins
8	Office floor plate, level 2, 75% glazing failure	51 mins	36 mins
9	Whole floor plate, level 2, Eurocode 1 fire load 80%	46 mins	49 mins
10	One tenancy, level 2, Eurocode 1 fire load 80%	46 mins	38 mins
11	1/3 of a tenancy, level 2, Eurocode 1 fire load 80%	46 mins	37 mins
12	Office floor plate, level 2, Eurocode 1 fire load 80%	46 mins	35 mins
13	Whole floor plate, level 2, Reduced lining factor	52 mins	54 mins
14	One tenancy, level 2, Reduced lining factor	51 mins	49 mins
15	1/3 of a tenancy, level 2, Reduced lining factor	51 mins	47 mins
16	Office floor plate, level 2, Reduced lining factor	51 mins	37 mins
17	Foggo tenancy, Level 2, BASE	51 mins	49 mins
18	Foggo tenancy, Level 2, 75% glazing failure	54 mins	57 mins
19	Foggo tenancy, Level 2, Eurocode 1 fire load 80%	46 mins	38 mins
20	Foggo tenancy, Level 2, Reduced lining factor	51 mins	49 mins
21	1/3 of tenancy, Lev 2, Centre, BASE	67 mins	71 mins
22	1/3 of tenancy, Lev 2, Centre, 75% glazing failure	86 mins	88 mins
23	1/3 of tenancy, Lev 2, Centre, Eurocode 1	60 mins	65 mins
24	1/3 of tenancy, Lev 2, Centre, Reduced lining fact	67 mins	71 mins

In some of the scenarios (for the Graphical Method and the Direct Method) the amount of ventilation that was available was above the upper limit of applicability of the equations. The amount of ventilation used by the computer program was therefore artificially reduced in order to keep it within its range of applicability. This effectively meant that the scenarios that were analysed were based on only a proportion of the glazed enclosure failing. At the high levels of ventilation that were available it would be a fuel load controlled fire (as opposed to a ventilation controlled fire) and further increasing the ventilation would result in less heat being retained within the enclosure and so a reduction in the fire severity. The artificial reduction in the ventilation that has been carried out would therefore increase the severity (i.e. the analysis will over-predict the fire severity).
The results of the analysis predict that the severity of the fire would require an equivalent (BS 476 test) fire protection to the structure in the order of 35 to 88 minutes. Therefore for the internal structure above deck level an equivalent fire rating of 90 minutes should be sufficient.

4.3 Secondary Beams

The structural engineers have allowed for a degree of redundancy in the layout and design of the secondary beams supporting the floors. It is such that localised failure of one beam and the

associated redistribution of load to adjacent beams would not cause structural collapse. Given the other assumptions in the structural fire strategy (specifically the 550°C temperature limit), this requires that the utilisation ratio of the adjacent beams does not exceed approximately 50%.
It is therefore proposed to increase the reinforcement of the slab and fire-rate only 50% of the secondary beams, providing fire protection to every other beam. In the event of a fire located below an unprotected structural beam, the beam may be exposed to temperature sufficient to cause it to fail, however, the next beam on each side would be fire rated and this, coupled with the enhanced slab, would ensure that the integrity of the floor remains.

5 EXTERNAL STRUCTURE

5.1 Scenarios

A separate analysis was carried out for the external structure. This structure would be outside the fire compartment itself and so would only be affected by any flames and radiation that project out of the windows. The severity of this would normally be significantly lower than for the internal structure.
Using the same scenarios as in the above analysis the effects of external flaming and radiation upon the external structure was analysed. Of the differing scenarios from above, those that produced the highest external flame temperatures were used to analyse the effect on the external members.

5.2 Flame Projection

The external flaming was calculated using guidance laid out in PD 7974: Part 3. The study into the effects of flame projection produced the following results, refer to Table 2. The results show that there is so much ventilation that combustion is able to take place completely within the floor plate and that flames would not project out of the windows. Any external structure would not experience any flame impingement, however it would receive radiation from the fire within the compartment. The compartment fires in numerous scenarios reached temperatures of over 1300°C and as such the radiation received by the external members would be likely to heat the structure to such a degree that fire protection would be required.

Table 2. Analysis of Flame Projection

Scenario	Flame projection
Full glazing failure	
Full floor area, north elevation	N
Full floor, west elevation, corner	N
Full floor, west elevation, centre	N
1/3 of floor area (one tenancy)	N
1/3 of a tenancy	N
100m2 office space	N

Scenario	Flame projection
75% glazing failure	
Full floor area, north elevation	N
Full floor, west elevation, corner	N
Full floor, west elevation, centre	N
1/3 of floor area (one tenancy)	N
1/3 of a tenancy	N
100m2 office space	N

5.3 Details of Analysis

An analysis was carried out to determine whether the external elements need any fire protection and if so the amount of fire protection that they would require. The structural elements investigated include the X-frames on the east and west façades and the Circular Hollow Sections supporting the north and south external bracings. This analysis consists of a number of stages:

a) The time/temperature curve of the compartment fire was taken as the one of the worst-case scenario (scenario 22) calculated for the internal structure protection analysis using Graphical Method. This time/temperature curve was then reduced by a factor corresponding to the losses during the radiation process through the air. As a result, the worst fire scenario would have a maximum air temperature of 1063°C on the heated surface of the external structure. The fire then burned for a total of 22 minutes before running out of fuel.
b) Computer program SAFIR was used to calculate the effect of fire on the external structural members. The effects of radiation only were analysed upon the elements of structure as it

was shown that there would be no flame engulfment. If the steel temperature was lower than 550°C during the worst fire scenario, no fire protection will be required. In the case where the steel had a temperature higher than 550°C, fire protection would be required.

c) For the structural element which requires fire protection, the required thickness of insulation was calculated using SAFIR to insure that the element would survive the worst fire scenario.

d) The same structural element with the same thickness of insulation was then analysed using BS 476 furnace fire and the time at which the steel reaches 550°C was calculated. This would give the equivalent severity of the predicted fire in terms of duration within the BS 476 test.

5.4 Results

The study indicates that there will be no flame impingement upon the external structure and would mean that the only heating the steel would undergo would either be from radiation from the compartment fire or a localised fire next to the external structure. Therefore the external structure is exposed to the radiative heat flux on one side only. The analysis result in recommendations on the amount of fire protection required on various external structural elements, as summarised in Table 3. Since the external structure is only exposed to heat on one side, only the surface facing the building needs to be protected if the structural element requires any fire protection.

Table 3. Summary of Fire Protection Required on Various External Structural Elements

External Structural Elements	Fire protection required when tested to BS 476
X Frame – 500x500x50 fabricated box section	Not required
X Frame – 400x400x16 SHS	41 minutes
X Frame Joints – 100mm plate	Not required
X Frame Compression bars – 711x40 CHS	45 minutes
X Frame Compression bars – 752x35.5 CHS	43 minutes
X Frame Tension bars – 550x300 solid bar	Not required
External Perimeter Bars – 500x200x16 RHS	45 minutes
External Elements – 457 CHS with thickness varying between 8mm to 25mm	42 minutes

6 CONCLUSIONS

An analysis has been carried out looking at fires within the various office areas in the building to determine whether a reduction in fire protection from the standards laid out in Approved Document B would be achievable. The intention of the analysis is to determine the likely level of fire protection that would be required to comply with the functional requirements of The Building Regulations. The amount of fire protection was designed to ensure that there would be no significant structural failure even in the event of the most onerous fire that could reasonably occur.

REFERENCES

[1] Approved Document B – Fire Safety, 2000 Edition, *TSO*.

[2] BS 476: Fire Tests on Building Materials and Structures, *British Standard.*

[3] The Building Regulations 2000, DCLG, *TSO*.

[4] BS 5950: Structural use of steelwork in building. Part 8: Code of practice for fire resistant design, *British Standard*, 1990.

[5] Eurocode 1: Actions on Structures. Part 1.2: General Actions - Actions on Structures Exposed to Fire, *European Committee for Standardisation (CEN)*, 2002.