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# COST TU0904

## Integrated Fire Engineering and Response

# Case Studies

March 2012

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**RESIDENTIAL BUILDINGS**



**REFURBISHMENTS**



**BRIDGES**

## PREFACE

Current practice in the European Union is safety, including Fire safety, nationally managed, and the demands are determined by the specific experiences of each country. While the political motivations for this approach are obvious, and local circumstances vary between countries, it can easily lead to similar processes having to be re-researched and re-invented country by country. In the context of the European Union safety requirements in case of fire are based on the Construction Products Directive 89/106/EEC. The Directive is applied to construction products as the essential requirement in respect of construction works. In Annex I of the Directive, the essential requirements for mechanical resistance and stability, and for fire safety, are summarised. The construction works must be designed and built in such a way that, in the event of an outbreak of fire: The load-bearing capacity of the construction can be assumed for a specific period of time; The generation and spread of fire and smoke within the works are limited; The spread of the fire to neighbouring construction works is limited; Occupants can leave the works or be rescued by other means; The safety of rescue teams is taken into consideration. The load-bearing capacity of the construction may be modelled on the principles summarised in the parts of the structural Eurocodes which deal with fire. The introduction of common standards in areas related to fire safety, it seems obvious that in such an important area the sharing of experience and research should be facilitated, and hence the need for networks in the COST model. For member states of the European Union,

However, the need for integration has a further dimension. Fire engineering researchers tend to specialise in areas such as fire dynamics, structural fire engineering, active/passive fire protection, environmental protection or human response. Since the background sciences of these disciplines are different there is little interaction between them. Practitioners, including fire engineers, building/fire control authorities, and fire-fighters tend to consider fire safety as a whole, but lack in-depth awareness of recent advances in research and are outside the academic research networks. Through encouraging the exchange of information on different aspects of fire engineering and response between researchers in different countries, the network intends to create an awareness of the current state of the art, and to avoid repetition of research. The non-research community will benefit from exposure to advanced research findings, discussion with researchers, and the sharing of best practice. Their input will make researchers aware of real-world constraints, and where new research and standards are needed.

The Action divides its membership loosely into three themed Working Groups, although clearly its overall mission of promoting integration means that these groups must interact on many of the key activities. The Working Groups are: WG1 Fire Behaviour and Life Safety focuses on the behaviour and effects of fire in buildings, combining this research-based knowledge with the most effective means of protecting human life against the occurrence of fire in the built environment. This includes active measures in fire-fighting with the effects of building form on the inherent risk to inhabitants. WG2 Structural safety covers the response of different building types to fires and the rapidly developing research field of structural fire engineering, including new materials and technologies and passive protection measures. Crucial problems of structural fire engineering concern change of use of buildings and the current imperatives of sustainability, energy saving and protection of the environment after fire. WG3 Integrated Design brings together design, practice and research across the disciplines of fire in the built environment. In structural design this includes integration of fire resistance with all the other functional requirements of a building, from concept onwards, rather than simply adding fire protection after all other processes are complete. Active input from practitioners, regulators and fire-fighters through this group is vital to the success of the Action.

The Action started in March 2010, and now has 22 nations of the EU and New Zealand participants. Its first deliverable, State of the Art Report attempts to bring together the current state of research, mainly in the participating countries but set into the context of knowledge world-wide. The second deliverable allowed all experts in Action to inform about its research findings in Proceedings and during the Action Prague Conference 29 April 2011 was focused outside the Action as well. This third deliverable Case Studies presenting current practice and accumulated knowledge in fire engineering. The authors, experts of the Action, are trying to include on the selected fire engineering applications clear explanations of the decision processes, the scientific assumptions and the practical constraints, as well as how different aspects of fire engineering are integrated.

František Wald and Ian Burgess

1 April 2012

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# 1 APPLICATION OF STRUCTURAL FIRE ENGINEERING TO THE TOWERS OF THE COURTHOUSE OF NAPLES

Summary

Fire Safety Engineering (FSE) is a multi-discipline aimed to define the fire safety strategy for buildings in fire situation, in which structural stability and control of fire spread are achieved by providing active and/or passive fire protection systems. In the following the main aspects of FSE for the structural safety checks in case of fire (Structural Fire Engineering) are shown with reference to Italian and European standards.

FSE requires the choice of performance levels, the definition of design fire scenarios, the choice of fire models and, generally, advanced thermo-mechanical analyses. In the following the application of Structural Fire Engineering (namely the structural behaviour in fire situation) to the existing building of the New Courthouse of Naples will be described. This activity is still in progress; nevertheless, the paper provides enough information concerning the structural characteristics of the building, the choice of safety performance levels, the active and passive protection systems of the building, the identification of fire scenarios through Risk-Ranking approach and, finally, preliminary thermal and structural analyses.

## 1.1 INTRODUCTION

According to ISO/TR 13387-1, the "Fire Safety Engineering" (FSE) is the application of engineering principles, rules and expert judgement based on a scientific assessment of the fire phenomena, the effects of fire and both the reaction and behaviour of peoples, in order to:

- save life, protect property and preserve the environment and heritage;
- quantify the hazards and risks of fire and its effects;
- evaluate analytically the optimum protective and prevention measures necessary to limit, within prescribed levels, the consequences of fire.

Current Italian and European codes (Ministry of Infrastructure and Transport, 2008, EN 1991-1-2 and EN 1992-1-2) allow the use of a performance approach through the concept of Fire Safety Engineering. The temperature distribution within the elements and the mechanical and geometric nonlinear structural response are taken into account in the fire performance approach.

The Directive 89/106/CEE on Construction Products of the European Community introduced the definition of the requirement of “safety in case of fire” in Europe, which is the base for the application of the Fire Safety Engineering. This requirement, implemented in the National Codes of European member countries, is explained by achieving the following five objectives:

- the load-bearing capacity of the construction can be assumed for a specific period of time;
- the generation and spread of both fire and smoke within the works is limited;
- the spread of fire to neighbouring construction works must be limited;
- occupants have to be able to leave the works or be rescued by other means;
- the safety of rescue teams must be taken into consideration.

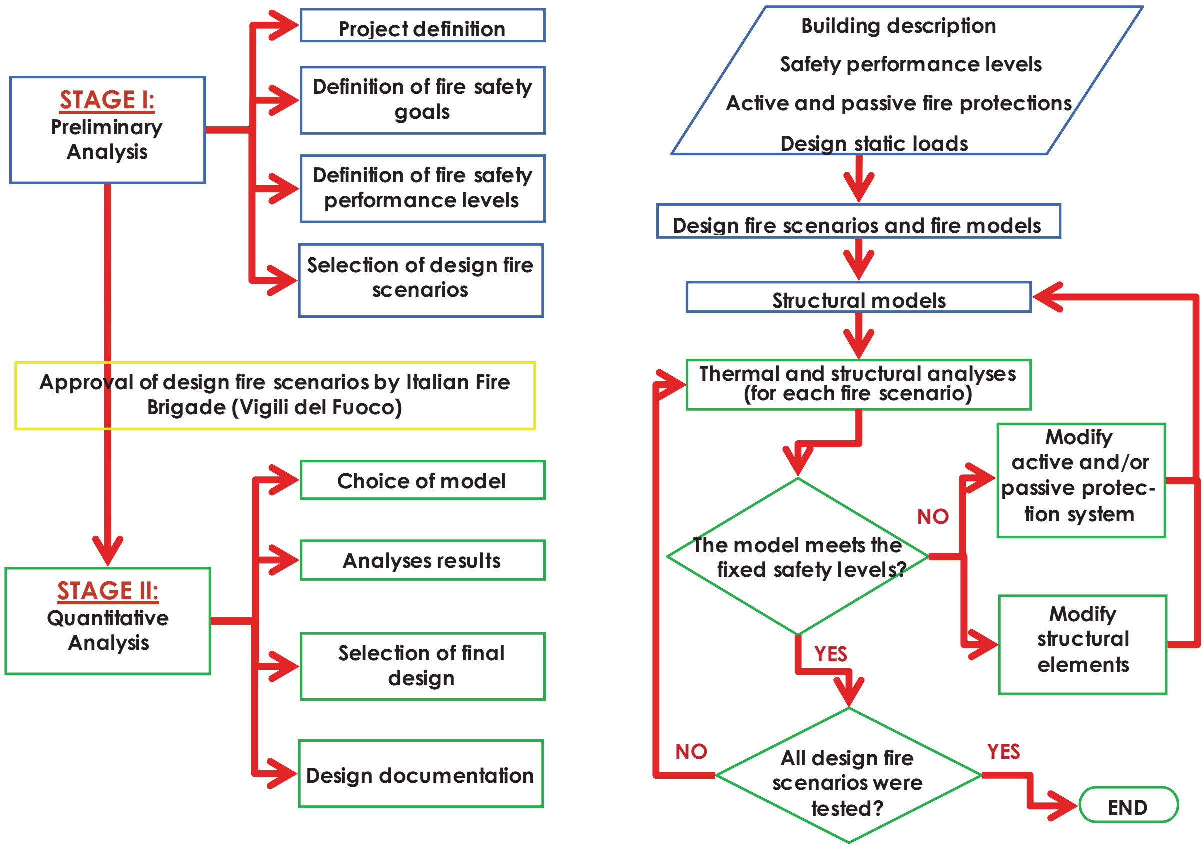
The results of each application of the performance approach to the fire safety should be evaluated through the analysis of the achievement of these objectives.

The Fire Safety Engineering allows a more accurate adjustment of the safety measures at specific risk of the building through qualitative and quantitative criteria (namely acceptance criteria), which are agreed with the building approval authority and hence form an acceptable starting point for assessing the safety of a building design.

The European codes for structural fire safety are the “Fire Parts” of Structural Eurocodes.

In Italy, the new Technical Code for Constructions was published in 2008 (Ministry of Infrastructure and Transport, 2008). For the first time in Italy, the fire action is introduced within the definition of the actions on constructions, as an “exceptional load”. The document defines the performance safety levels of buildings according to the safety objectives required by the Directive 89/106/CEE (Construction Product Directive, 1988). The Italian Technical Code for Constructions defines five safety performance levels depending on the importance of the building, which establish the damage level that can be accepted. These rules define the fire structural performance requirements and they refer to specific technical codes issued by the Italian Ministry of Interior for all activities under the control of the National Fire Brigades (Ministry of Interior, 2007a and Ministry of Interior, 2007b). The regulations are basically prescriptive and concern several types of building use. However, the performance-based fire design and advanced calculation models may be applied either in the lack of prescriptive rules or in the case of “derogation” with respect to prescriptive rules. The performance-based design (or engineering approach) has to be developed according to Decree of the Ministry of the Interior of 09/05/2007 (Ministry of Interior, 2007b), titled “Direttive per l’attuazione all’approccio ingegneristico alla sicurezza antincendio”. The fire design, according to D.M.

09/05/2007, summarized in Fig. 1.1, is divided in two stages: the first one is preliminary analysis, i.e. qualitative analysis, while the second one is quantitative analysis. Between the first and second stage, the approval of design fire scenarios by Italian Fire Brigades (Vigili del Fuoco) is needed. Finally, it is important to note that in the current Italian Codes the performance-based approach does not replace the prescriptive one, but both the approaches coexist. The technical solutions imposed by the prescriptive approach remain one of the possible ways that the designer may choose for the structural fire design.




(a) (b)

Fig. 1.1 Fire Safety Engineering: Italian code process according to Decree of the Ministry of the Interior of 09/05/2007 (Ministry of Interior, 2007b)

### 1.2 CASE STUDY: TOWER "A" OF THE COURTHOUSE OF NAPLES

In the following the application of Structural Fire Engineering (namely the structural behaviour in fire situation) to the existing building of the New Courthouse of Naples is described. The latter, located in the administrative centre of Naples (Italy) and intended for office use, is divided into three main areas, namely Lot 1, 2 and 3, built on a reinforced concrete foundation system (located at 11.45 m above the sea level). In the central part of the construction (corresponding to the Lot 2), three towers of different heights rise from a large area (named "covered square") situated at an altitude of 18.30 m above sea level. The lower tower

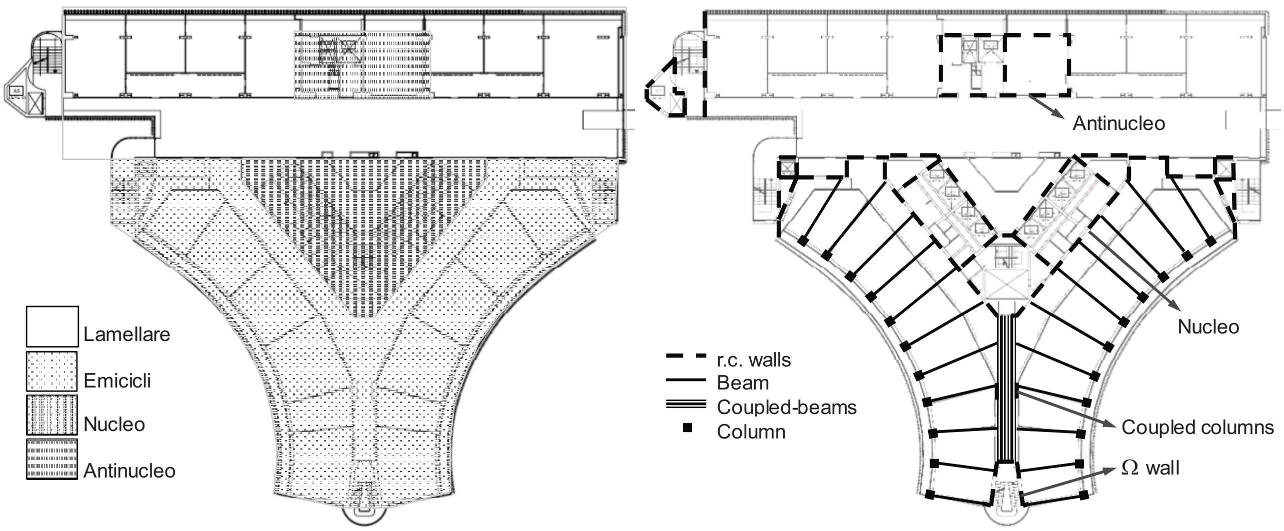
(Tower C), located on the east side, extends from a height of 30.00 m to a height of 69.60 m above sea level; the intermediate one (Tower B) develops from a height of 30.00 m to a height of 89.40 m above sea level, the highest one (Tower A) extends from a height of 30.00 m to a height of 112.50 m above sea level. The Towers, with 17, 23 and 29-storeys, respectively, are characterised by reinforced concrete central cores and, from 30.00 m above sea level, perimeter steel beams and columns. These latter are protected by several passive protection systems.

In the following the attention will focus only on the highest tower (Tower "A").

### 1.2.1 Building description: analysis of the structural characteristics

The Tower A is 101.00m high and has 29-storeys above the ground (see the left side tower in Fig. 1.2). The floor can be divided into four zones, named (see Fig. 1.3a): 1) Lamellare, 2) Emicicli , 3) Nucleo, 4) Antinucleo. In particular the third and fourth zone, made of reinforced concrete, represent the bracing and seism-resistant structures of the Tower at each floor. Other stiffening reinforced concrete structures (Fig. 1.3b) are: stairwells, omega wall and coupled columns. Until 30.00 m above sea level the bracing structures are connected to a reinforced concrete framed structure, having large beams and columns, whereas, from 30.00 m above sea level, for 25 storeys, the bracing structures are connected to steel frames having an interstorey height equal to 3.30 m.

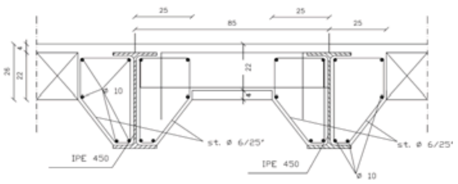
Fig. 1.2 New Courthouse of Naples: South side view



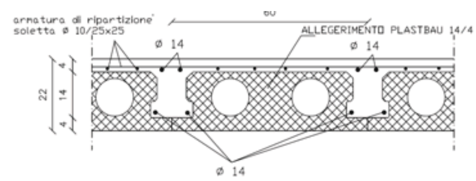

(a) (b)

Fig. 1.3 New Courthouse of Naples: (a) Floor Map, (b) Structural elements

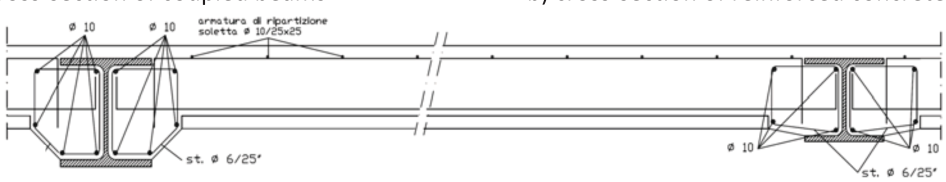
Referring to “emicicli” zone (see Fig. 1.3a), from 30.00 m above sea level, there are primary steel beams arranged in a radial pattern, which join the exterior steel columns to the reinforced concrete wall of the “nucleo” zone or to the coupled beams (which join the “Ω wall“ to the “nucleo” zone wall, see Fig. 1.3a). All members are connected by pinned joints as shown by the construction details reported in Fig. 1.4d,e,f. The coupled beams with IPE450 steel profile are partially encased with concrete (see Fig. 1.4a). The primary steel beams, arranged in a radial pattern, are also partially encased with concrete and have several cross-section dimensions as a function of span length. In particular, there are four types of cross-section, with steel profile HEB 240, HEB 260, HEB 300 or HEB 340 (see for example Fig. 1.4c). The floor deck, with an overall depth equal to 220mm and superior concrete slab equal to 40mm, are reinforced concrete members with lightweight polystyrene blocks. The secondary beams are IPE 180 steel profile. The steel columns are square hollow steel section 350x350mm$^2$ with thickness varying between 10 mm and 20 mm along the height; in Tab. 1.1 the columns steel section along the height are summarized.



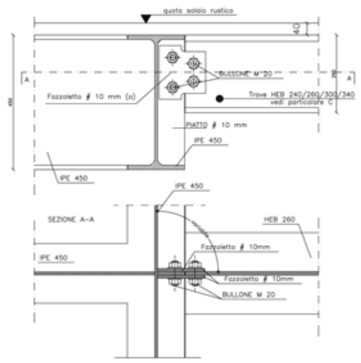

a) cross-section of coupled beams



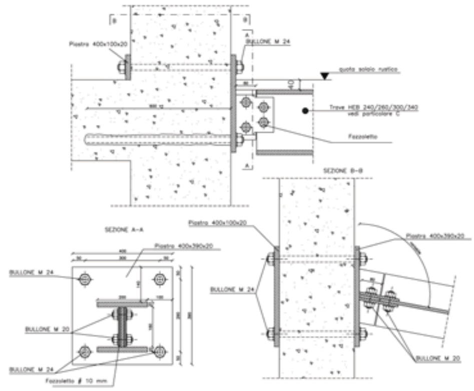

b) cross-section of reinforced concrete slab



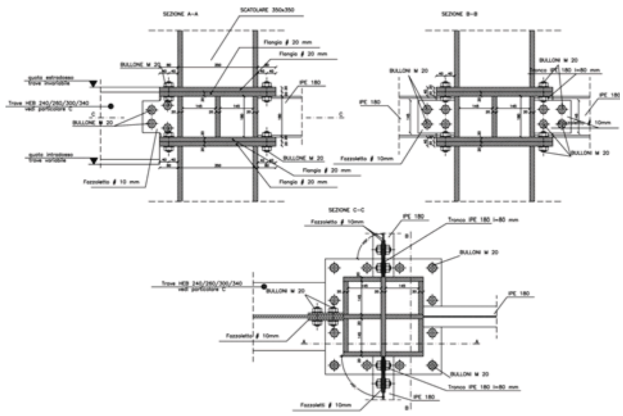

c) cross-section of primary beam and longitudinal-section of reinforced concrete slab



d) joint between primary beam and coupled beam



e) joint between primary beam and RC wall



f) joint between steel beam and steel column

Fig. 1.4 Construction details

Tab. 1.1 Columns' cross-sections

| Height (m above sea level) from | Height (m above sea level) to | hollow steel section mm x mm x mm | Number of columns |
|---|---|---|---|
| 30.00 | 39.90 | 350x350x12.5 | 8 |
| | | 350x350x16 | 14 |
| 39.90 | 49.80 | 350x350x12.5 | 12 |
| | | 350x350x16 | 10 |
| 49.80 | 59.70 | 350x350x10 | 10 |
| | | 350x350x12.5 | 12 |
| 59.70 | 69.60 | 350x350x10 | 14 |
| | | 350x350x12.5 | 8 |
| 69.90 | 112.50 | 350x350x10 | 22 |

**1.2.2 Choice of safety performance level**

In the case study, the main objective of fire safety checks concerns the mechanical resistance and stability, in fire situation, of the tower. In agreement with the Fire Brigades and Owner, the safety performance level required for the structure is assumed as: "*maintaining the fire resistance requirements, which ensure the lack of partial and/or complete structural collapse, for the entire duration of the fire*".

In addition, with reference to some scenarios (the most probable fire scenarios which involve the effectiveness of active protection systems), a limited structural damage after the fire exposure has been also required.

### 1.2.3 Active and passive fire protection systems

The tower is equipped with several active protection systems: fire sprinkler system, fire hydrants and fire extinguishers. The building is not equipped with any smoke or heat evacuation systems. Each floor of the tower have 4 fire exits on external stairways and 1 fire exit on internal separated stairways equipped with 2 fire doors REI 120. Each floor can be divided in 3 fire compartments (see Fig. 1.5).

Both steel beams and columns are protected by gypsum boards.

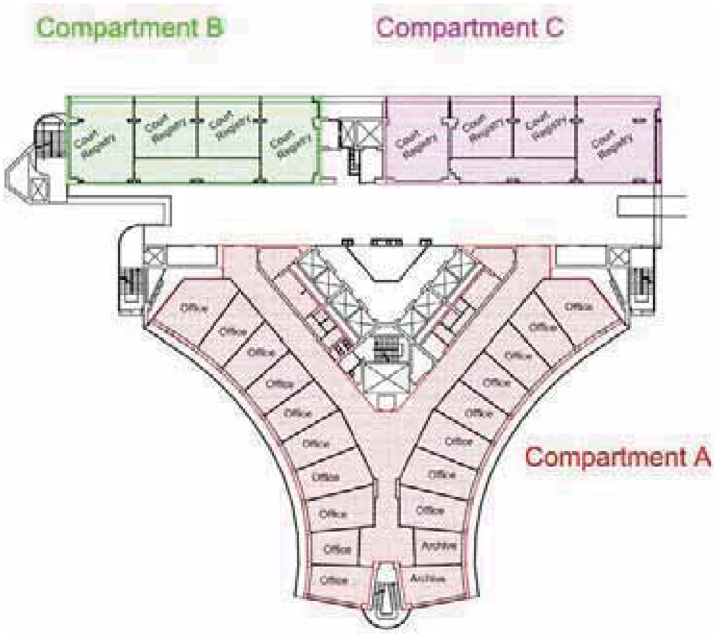



Fig. 1.5 Fire Compartments

### 1.2.4 Static and fire design load calculation

The Italian and European codes (Ministry of Infrastructure and Transport, 2008 and EN 1991-1-2) classify the fire as an exceptional load, so the design load combination in fire situation is defined by:

$$F_d = A_d + G_{k1} + G_{k2} + \sum_{i=1}^{n} \psi_{2i} \cdot Q_{ki} \qquad (1)$$

where $G_{k1}$ is the characteristic value of structural permanent load; $G_{k2}$ is the characteristic value of non-structural permanent load; $\psi_{2i} \cdot Q_{ki}$ is the quasi-permanent value of a variable action *i*; $A_d$ is the design value of the exceptional action (fire).

The compartment's fire load density is closely linked to actual combustible contents of the building or rooms and, therefore, it is depending on the building or room occupancy. In the case study the value of fire load density is based on fire load classification of occupancies provided by EN1991-1-2 (2004). Therefore, according to office use for the building, the characteristic fire load density $q_{f,k}$ [MJ/m²] is assumed equal to 511MJ/m$^2$ (80% Fractile), as given in Table E.4 of EN 1991-1-2 (2004).

### 1.2.5 Fire Scenarios and fire models

The design *fire scenario* is a qualitative description of the fire development during the time, identifying key events that characterise the fire and differentiate it from other possible fires. It typically defines the ignition and fire growth process, the fully developed stage, decay stage together with the building environment and systems that will impact on the course of the fire.

In general, the number of distinguishable fire scenarios is too large to permit analysis of each one. In this case the choice of the design fire scenarios is carried out by *Fire Risk Assessment*. Really, the Fire Risk Assessment allows to individuate scenario structures of manageable size and allows to make the case that the estimation of fire risk based on these scenarios is a reasonable estimation of the total fire risk (0). The Fire Risk Assessment takes into account the consequence and likelihood of the scenario. Key aspects of the process are:

- identification of a comprehensive set of possible fire scenarios;
- estimation of probability of occurrence of each fire scenario;
- estimation of the consequence of each fire scenario;
- estimation of the risk of each fire scenario (combination of the probability of a fire and a quantified measure of its consequence);
- ranking of the fire scenarios according to their risk.

The Fire Risk Assessment is performed through the *event tree approach*, according to ISO-16732 Guidelines. A fire scenario in an event tree is given by a time-sequence path from the initiating condition through a succession of intervening events to an end-event. Each fire scenario corresponds to a different branch of the event tree, and the branches collectively comprise or represent all fire scenarios.

The following main events, that may affect the development of the fire, are considered:

- First aid suppression
- Alarm activation (smoke detectors)
- Sprinkler activation
- Sprinkler suppression
- Barrier effectiveness.

In Fig. 1.6 the event tree obtained combining the main events is reported.

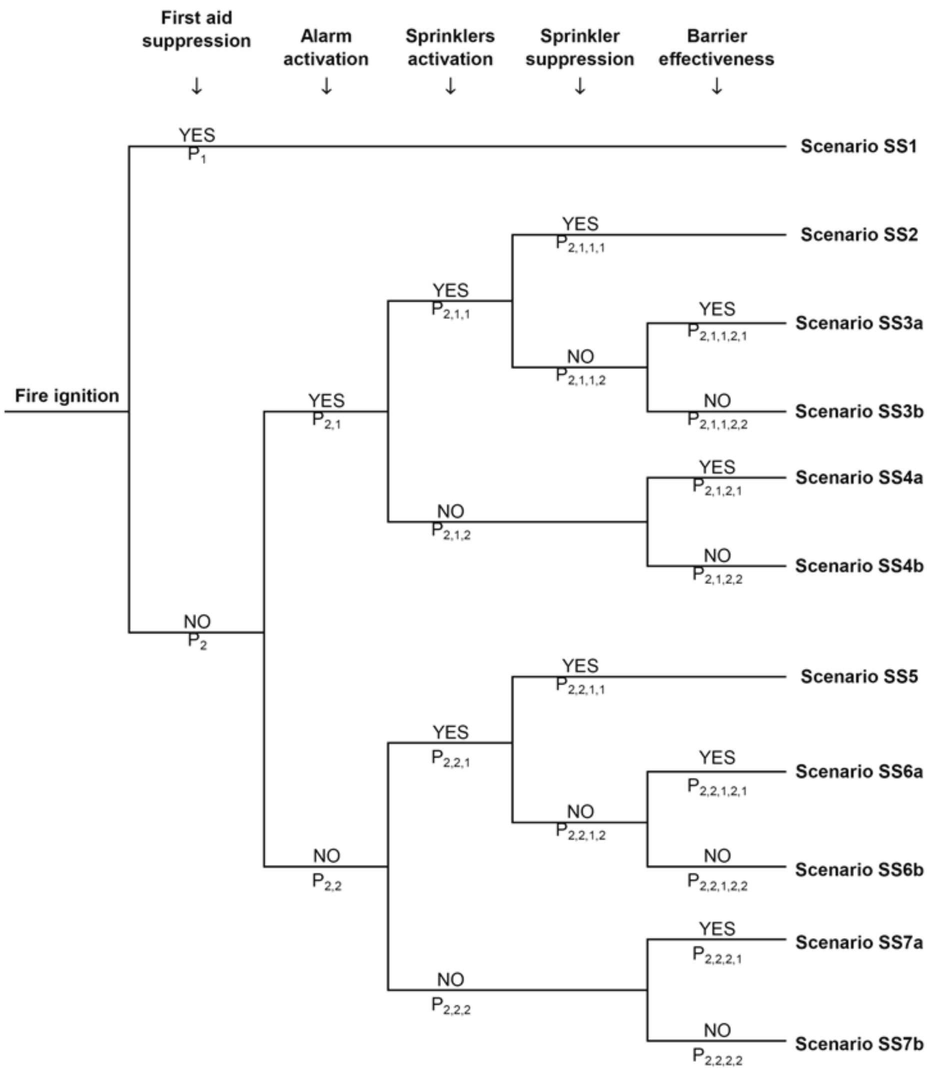



Fig. 1.6 Event tree

Probability of occurrence of each event and consequence value of each fire scenario is obtained both by direct estimation from available data (0, Hall, 2010, Nystedt, 2011 and Hasofer, 2010) and engineering judgment (see Fig. 1.7). The consequence value is expressed as a fraction of the economic value of the building. For each fire scenario the relative risk (R) is evaluated by multiplying the measure of the consequence (C) by the probability of occurrence of the scenario (P):

$$R = P \cdot C \quad (2)$$

Finally, in Tab. 1.2 the risk ranking is reported. The highest fire risk is for the Scenario SS7a , where:

- first aid suppression failed;
- alarm activation failed;
- sprinkler activation failed;
- barrier effectiveness.

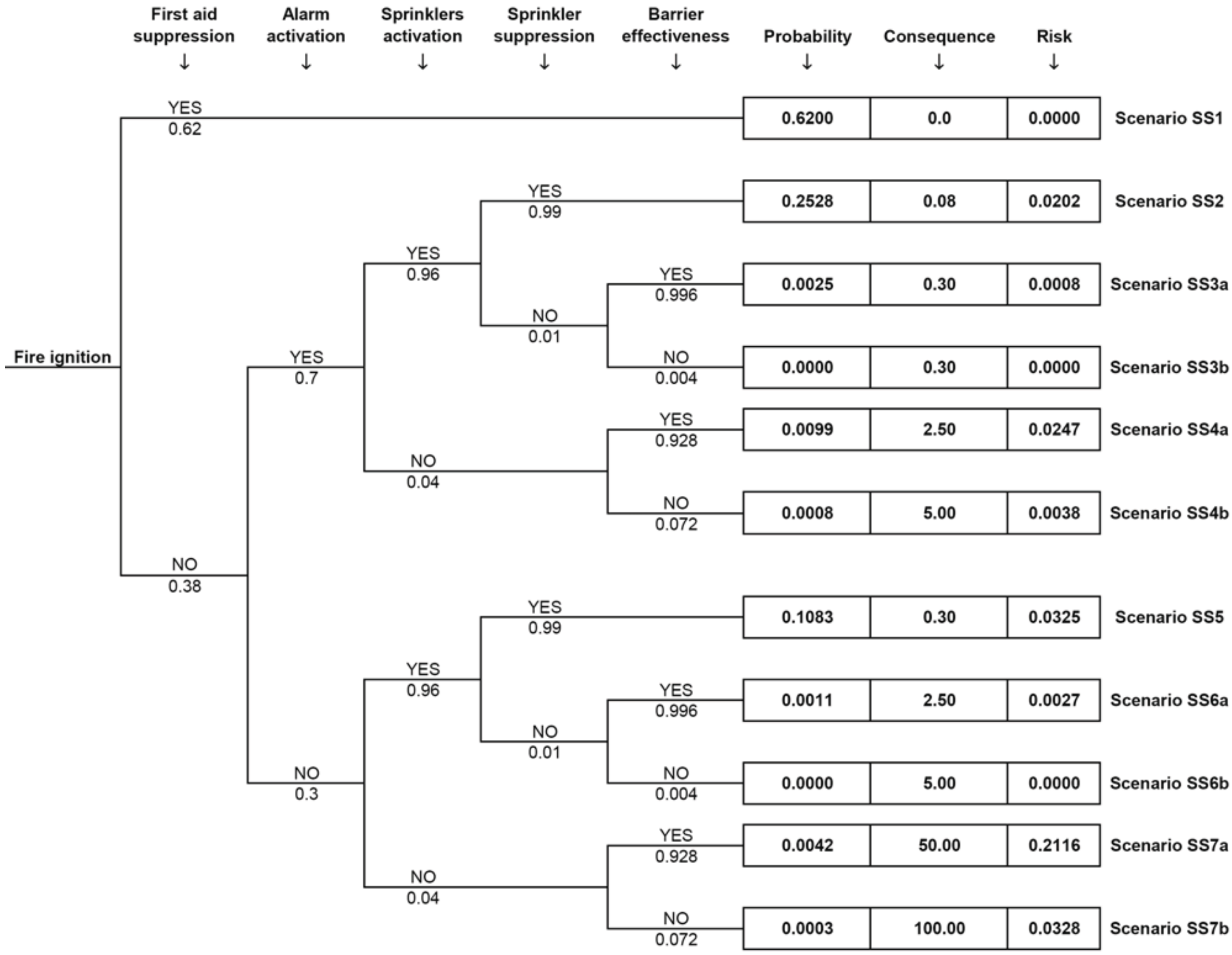



Fig. 1.7 Event tree

Tab. 1.2 Risk ranking

| Scenario | Probability | Consequence | Risk | Risk Ranking |
|---|---|---|---|---|
| Scenario SS1 | 0.6200 | 0.00 | 0.0000 | 11 |
| Scenario SS2 | 0.2528 | 0.08 | 0.0202 | 5 |
| Scenario SS3a | 0.0025 | 0.30 | 0.0008 | 8 |
| Scenario SS3b | 0.0000 | 0.30 | 0.0000 | 10 |
| Scenario SS4a | 0.0099 | 2.50 | 0.0247 | 4 |
| Scenario SS4b | 0.0008 | 5.00 | 0.0038 | 6 |
| Scenario SS5 | 0.1083 | 0.30 | 0.0325 | 3 |
| Scenario SS6a | 0.0011 | 2.50 | 0.0027 | 7 |
| Scenario SS6b | 0.0000 | 5.00 | 0.0000 | 9 |
| Scenario SS7a | 0.0042 | 50.00 | 0.2116 | 1 |
| Scenario SS7b | 0.0003 | 100.00 | 0.0328 | 2 |

Therefore, fire scenario SS7a is a design fire scenario: the structure is required to “maintain the fire resistance requirements, which ensure the lack of partial and/or complete structural collapse, for the entire duration of the fire”.

Moreover, another design fire scenario is fire scenario SS5, characterized by a higher probability of occurrence, for which limited damages are allowed for the structure.

Finally, also the following secondary events can be significant:

- doors state (open or closed);
- windows state (open or closed).

The state of the secondary events will be considered inside the fire model as well as the location of fire ignition.

The *post-flashover fire* is modelled by one-zone model, which assumes homogeneous temperature, density, internal energy and pressure of the gas in the compartment.

**1.2.6 Substructure identification by means of preliminary analyses**

In the case study, due to the building’s large size, in order to reduce the computational time the *substructure analysis* is adopted, according to Eurocode suggestions. Several preliminary analyses allow to define the substructures limits and boundary conditions. The aim of the substructure analysis is to evaluate the structural fire response through the modelling of significant parts of the entire structure. The designer has the responsibility to choose the substructure in such a way that the hypotheses on the constant boundary conditions are reasonable and correspond at least to a good approximation of the real situation (Franssen, 2005).

Preliminary analyses are carried out on a 25-storey plane frame extracted from the “Emicicli” zone (Fig. 1.9); this simplification is possible because the RC slab is designed as simply supported by primary beams, as shown in Fig. 1.4c. In these preliminary analyses the structural members are considered without protection systems. The analyzed frame has been chosen in order to analyze structural members with the maximum degree of utilization at time t = 0 ($E_{d,fi}/R_{d,fi,0}$). All members (beam-column and beam-concrete wall) are connected by pinned joints, as shown by the construction details reported in Fig. 1.4 e,f.

The preliminary analyses are carried out adopting the standard time-temperature curve ISO834, with the only purpose of defining the substructure, which should represent the global structural behavior: really, ISO834 curve allows a direct comparison in term of fire resistance time. Two fire positions are considered (see Fig. 1.9) in order to evaluate possible column’s buckling phenomenon due to fire scenarios localized on floors in which there is the change section of the columns.

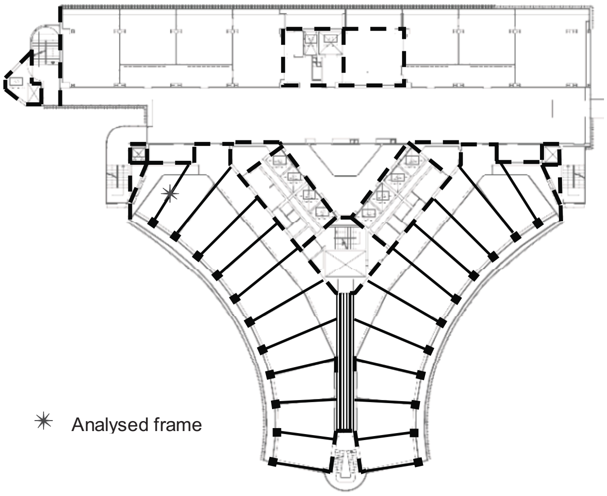



Fig. 1.8 The analysed frame

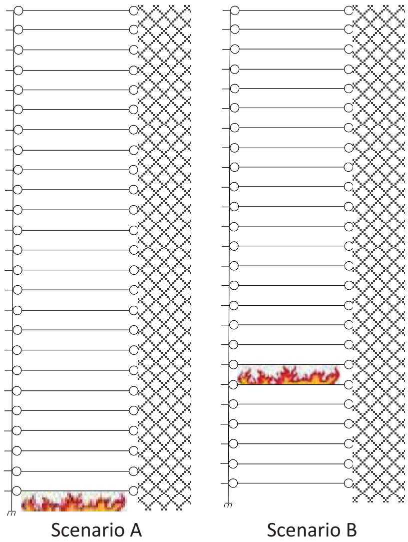



Fig. 1.9 Fire scenarios: possible position along height

Based on the following consideration is possible to define the potential substructure extension and boundary conditions. The extension of considered substructure (see Fig. 1.10) is made by the beam and column exposed to fire and by the cold column above the compartment, which contributes to translational and rotational constraint of nodes of exposed structure. Regarding the boundary conditions, the part of structure above the cold column keeps stiffness, so it's replaced by rigid restraint. Moreover, vertical displacements of cold column are allowed in order to transfer the loads from the above structure. Finally, the cold part of structure below the exposed compartment becomes stiffer than the heated part, so that it is replaced by rigid restraint.

The comparison between thermo-mechanical behaviour of considered substructures and the 25-storey plane frame (entire structure) one allows to evaluate the validity of the substructure, for which the thermo-mechanical behaviour is analysed with reference to natural fire curves. As previously said, post-flashover fire is modelled by one-zone model. Numerical fire analyses are performed by using the non linear software SAFIR2011 (Franssen, 2005), developed at the University of Liege (Belgium).

#### 1.2.6.1 Analyses results

The fire resistance time reported in Fig. 1.10 shows that the substructures (one for each fire scenario) are able to represent the global structural behaviour. The results clearly show that columns are the weakest element in the structure: in fact failure occurs due to the columns failure. In the preliminary analyses, the latter are unprotected thin square hollow steel sections (350mmX350mmX12.5mm for scenario A and 350mmX350mmX10mm for scenario B), while a concrete coating protects steel beams (HE260B) by fire exposure. Column, loaded with constant axial force during fire exposure, fails mainly due to buckling, that clearly occurs for reduction of steel stiffness and strength produced by heating.

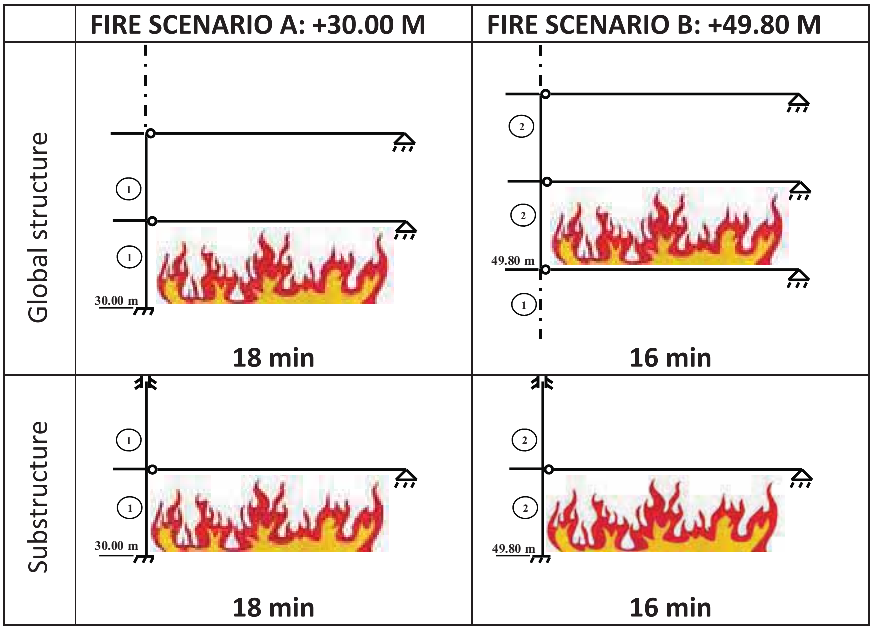



Fig. 1.10 Analyses results

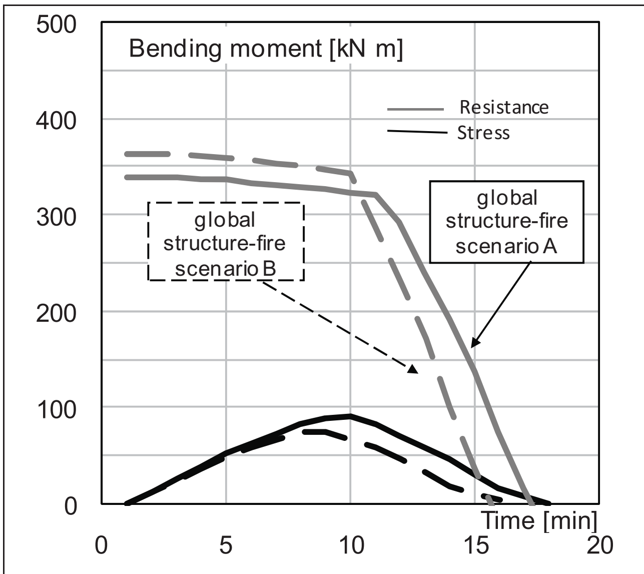



Fig. 1.11 Acting bending moment and Resistance capacity to combined compression and flexure on the heated column of global structure – comparison between fire scenario A and B.

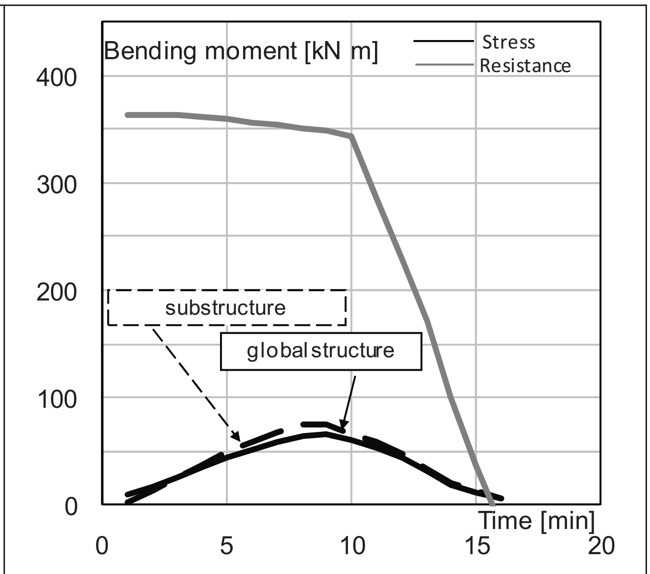



Fig. 1.12 Comparison between global structure and substructure results – Fire scenario B.

Accordingly, analyses results on global structure (see Fig. 1.11) show that the minimum fire resistance occurs when fire involves the thinnest column (fire scenario B). Really, the latter is characterized by a section factor ($A_m$/V) bigger than thickest column: the highest section factor produces a fast thermal degradation of the thinnest column. As concerns the comparison between substructure and global structure (see Fig. 1.12), approximately the same time of collapse is attained, because the stiffness of beams is not able to affect the axial force in the columns.

### 1.2.7 Thermo-mechanical analyses with reference to the selected fire scenarios

Subsequent analyses are carried out on substructure characterized by the thinnest tubular columns (350mmx350mmx10mm) and HE260B beams (partially encased with concrete). Both steel beams and columns are protected by gypsum boards.

As previously said, the scenario with the highest risk is Scenario SS7a for which:

- first aid suppression failed;
- alarm activation failed;
- sprinkler activation failed;
- barrier effectiveness.

#### 1.2.7.1 Fire Scenario SS7a - Fire model

Fire curve (see Fig. 1. 14) is obtained by one zone model (Cadorin, 2001). Fig. 1. 13 shows the Rate of Heat Release obtained in accordance with EN1991-1-2 (2004).

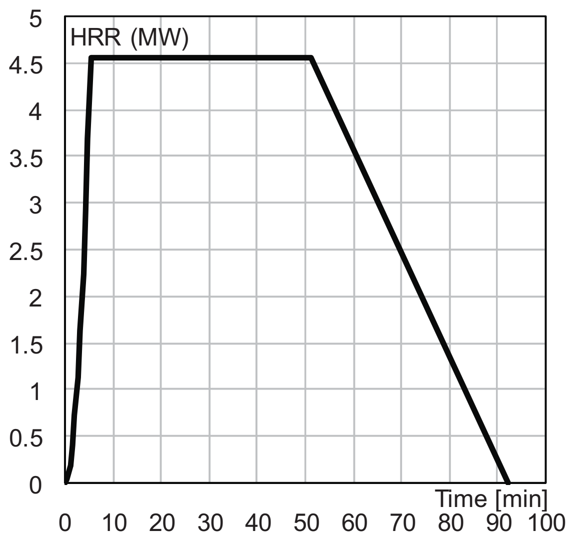



Fig. 1. 13 Rate of Heat Release

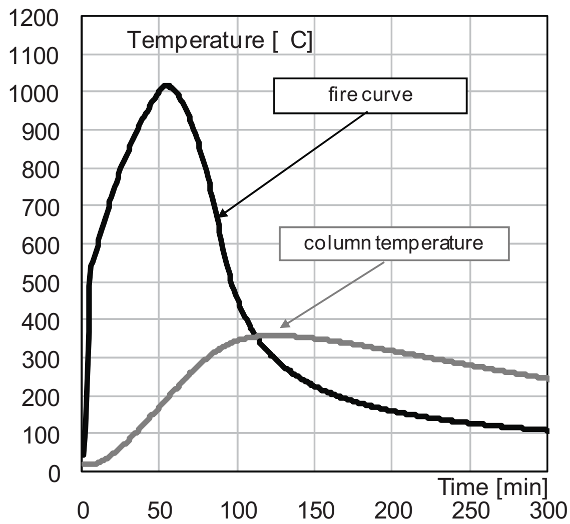



Fig. 1. 14 Comparison between fire curve and column temperature

#### 1.2.7.2 Fire Scenario SS7a - Structural Behaviour

Column's temperature is lower than 400°C during whole fire exposure time, as shown in Fig. 1. 14. Therefore combined axial and bending moment resistance is approximately constant and higher than design actions (see Fig. 1. 15).

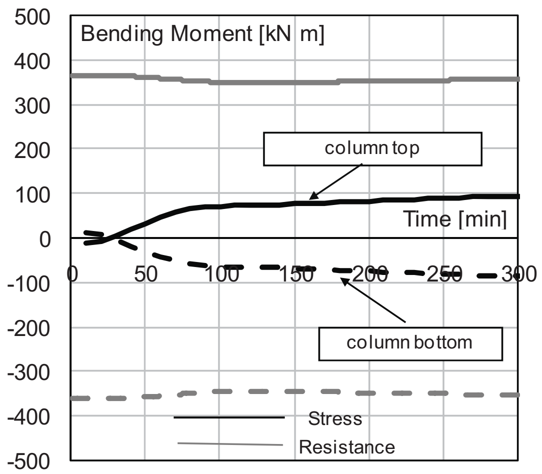



Fig. 1. 15 Bending moments and Resistance capacity to combined compression and flexure on heated column

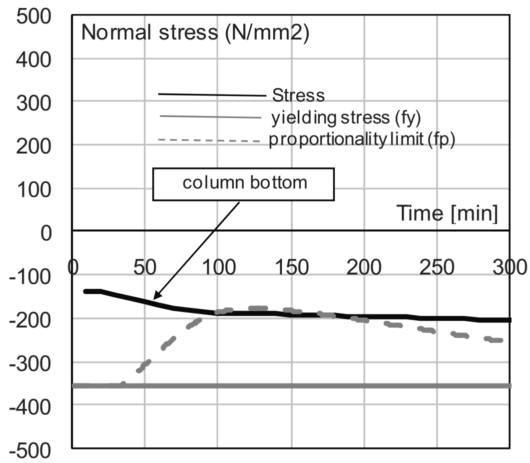



Fig. 1. 16 Comparison between normal stress, proportionality limit and yielding stress in heated column

Comparison between normal stress and yielding stress, during fire exposure time, shows that no significant plastic strains occur in the heated column: maximum normal stresses are slightly greater than proportionality limit between 100 min and 180 min (see Fig. 1. 16). Therefore SS5 scenario's analysis is not significant: in fact, in this fire scenario the sprinkler activation extinguishes the fire and the heat release rate decreases to zero after some decreasing time (Staffansson, 2010),0 see Fig. 1.17.

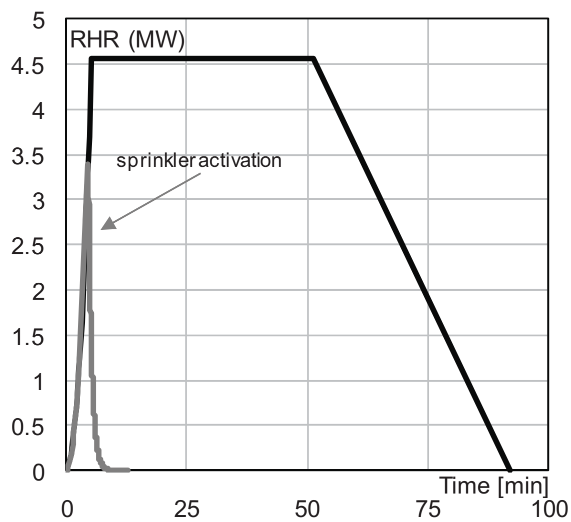



Fig. 1.17 RHR curves

### 1.2.8 Future developments

The activity presented above is still in progress. In future the fire development and its effects on the structure will be evaluated by a computational fluid dynamic model (i.e. FDS software), used to solve numerically the partial differential equations giving, in all points of the compartment, the thermo-dynamic and aero-dynamic variables.

The structural analyses will be carried out by several non linear softwares (SAFIR, ABAQUS and STRAUS7), with the aim of performing also detailed 3D thermo-mechanical analyses.

**1.3 CONCLUSION**

This paper is devoted to the application of Structural Fire Engineering (according to Italian and European Codes) to a tower of the Courthouse of Naples. The tower, with office use, is 101.00 m high and has 29-storeys above the ground; the main structure is realised with a reinforced concrete central core and perimeter steel beams and columns.

In the presented case study, the objective of fire safety assessment concerns the mechanical resistance and stability in fire situation of the tower. In agreement with Fire Brigade and building's Owner, the performance level assumed for fire safety check of the structure is: "maintaining of the fire resistance requirements, which ensure the lack of partial and/or complete structural collapse, for the entire duration of the fire". In addition, with reference to the most probable fire scenarios, which involve the effectiveness of active protection systems, a limited structural damage after the fire exposure is also required.

The identification of design fire scenarios is carried out by means of Fire Risk Assessment, applying the event tree approach according to ISO-16732 Guidelines. A fire scenario in an event tree is given by a time-sequence path from the initiating condition through a succession of intervening events to an end-event. Each fire scenario corresponds to a different branch of the event tree, and the branches collectively comprise or represent all fire scenarios. The main events taken into account in the risk assessment, that may affect the development of the fire, are: first aid suppression; alarm activation (smoke detectors); sprinklers activation; sprinklers suppression; barrier effectiveness. Moreover, the following secondary events can be significant: doors state; windows state. The state of the secondary events is taken into account inside the fire model as well as the location of fire ignition.

The post-flashover fire is modelled by one-zone model, which assumes homogeneous temperature, density, internal energy and pressure of the gas in the compartment, applying Ozone software.

In order to evaluate the structural fire safety, Italian and European Codes allow the global structural analysis, the analysis of part of the structure (substructure analysis) and the analysis of a member (single member analysis). In the case study, due to the building's large size, in order to reduce the computational time, the substructure analysis is adopted. The static scheme of the building allows to define simple substructures, which are able to represent the global structural behaviour. It should be noted that the structural static scheme doesn't produce significant indirect actions on columns.

The results of the structural analyses under the highest risk fire scenario (SS7a) show that both column and beam's temperatures are lower than 400°C during fire exposure (see Fig. 1. 14), thanks to passive protection systems: therefore, no relevant plastic strains occur in the structure (see Fig. 1. 16).

Accordingly, SS5 scenario's analysis is not significant: in fact, the sprinkler activation extinguishes the fire and the heat release rate decreases to zero after some decreasing time (see Fig. 1.17).

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# 2 HERON TOWER, LONDON
## Arup Fire Ltd

Summary

Heron Tower is a high-rise office building, recently constructed in the City of London, designed by architects Kohn Pederson Fox Associates for the property development group Heron International. The building provides over 68,000m$^2$ of floor space, comprising mainly offices with a small amount of retail at the ground and first floors. A restaurant and bar have been provided on the 38th to 40th floors, to be open to members of the public. The 47-storey tower rises to 203m in height, with a mast of 39m taking the highest point to 242m. Heron Tower was completed in 2010, and is one of the city's tallest buildings.

Fig. 2.1 Heron Tower - The completed building was opened in 2010

### 2.1 ARUP INVOLVEMENT

The project was run by Building Group 1 in London with Arup involvement on structures, acoustics, security, geotechnics, transportation, facades, IT and communications, as well as fire engineering. Arup Fire was involved in the project since its inception in 1999, initially to provide fire strategy advice up to the Planning Application, but its role subsequently grew to include CFD modelling, structural fire engineering and an extreme events study. The fire engineering design was largely completed in 2006 when conditional approval was granted by the City of London under Part B (Fire Safety) of the Building Regulations (2000) and Section 20 (Fire Safety in Section 20 Buildings) of the London Building Acts 1939 (LDSA 1997).

#### 2.1.1 Fire Engineering Strategy

A key requirement of the architectural design was to maintain an open, interconnected feel to the building. This has been achieved by subdividing the tower into ten 3-storey villages, each with accommodation arranged around a central atrium. Each 3-storey village is separated from the next by a 2-hour compartment floor; hence the principle behind the fire safety design was to treat each village as a 3-storey building connected by an open void. The building is also split vertically into two zones, with the accommodation and atria situated to the north of the building and the core zone, containing combined fire fighting / escape stairs and plant-rooms situated to the south. To increase its attractiveness to tenants, the client wanted complete flexibility of the villages to allow tenants either to enclose the atria or to leave them open to the accommodation. Because open atria would introduce a direct route for smoke to spread between levels, the fire safety design was developed using a simultaneous evacuation regime within each village, also ensuring that occupants on all parts of the floors can always escape away from the atrium in order to reach the escape cores.

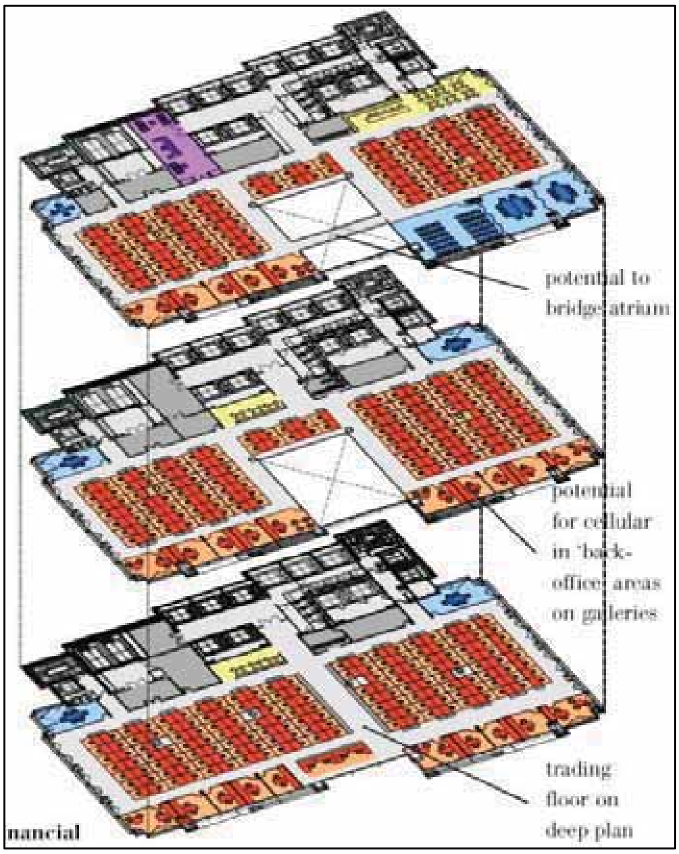



Fig. 2.2 Layout of a three-floor "village"

**2.1.2 Computational Fluid Dynamics**

The British Standards recommend that a smoke reservoir be provided in the top of the atrium to delay the time it takes for the smoke layer to build down to a level where it could spread back onto the upper floors and hence potentially affect escape. In this case, in order to create a suitable reservoir, it would have been necessary to separate the uppermost level of the atrium with smoke retarding construction. However, to achieve the flexibility of open or enclosed atria desired by the client, CFD modelling was

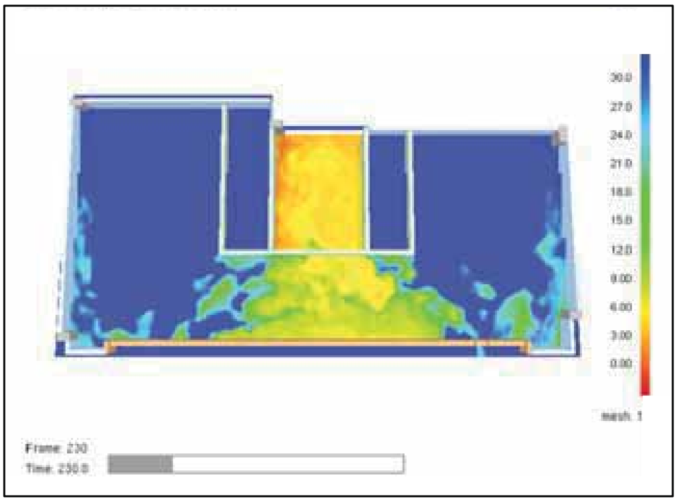


Fig. 2.3 CFD model of smoke spread from atrium to upper floor of a village

undertaken to demonstrate that occupant evacuation at the upper levels would not be compromised by the smoke spreading from a fire at one of the lower levels via the open sided atria.

The CFD analysis was run in two parts. The first model (Fig. 2.3) was created to assess the conditions that occupants of the top floor of a village may face as a result of smoke spreading via the atrium from a fire on a lower floor. An axi-symmetric plume in the base of the atrium and a spill plume from the lowest level were modelled. It was demonstrated that for both scenarios, occupants would have adequate time to evacuate away from the atrium and into cores before the onset of untenable conditions due to visibility, temperature and carbon monoxide levels.

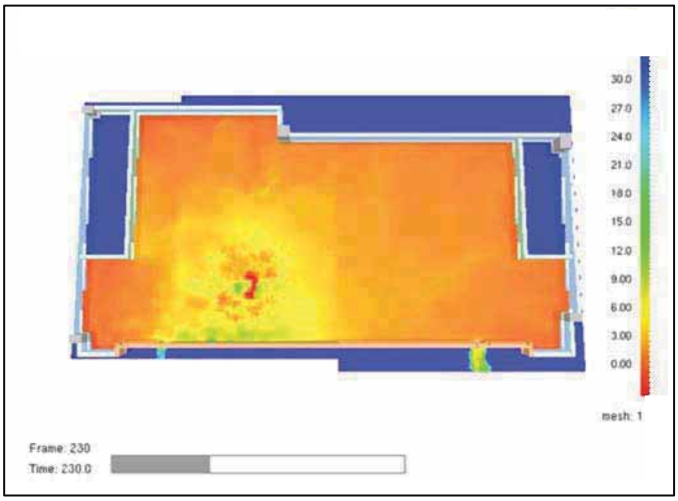


Fig. 2.4 CFD model of smoke conditions on a floor without an atrium

The second model was created to assess the conditions occupants might face on a single floor of the building if there was no atrium, i.e. a possible 'code compliant' arrangement. The results of this analysis demonstrated that conditions would be significantly better in the proposed village arrangement with an atrium when compared to a single storey arrangement without an atrium.

It was therefore demonstrated that the village concept would not compromise occupant life safety due to smoke spread, and that the design performed better than a possible code compliant arrangement. Close consultation with the District Surveyor early on in the design resulted in a smooth approvals process when the modelling results were presented. This was a key milestone for the client and provided confidence that the village concept would be acceptable.

## 2.2 STRUCTURAL FIRE ENGINEERING

The main superstructure of Heron Tower is a Vierendeel stress tube that wraps around the perimeter of the office floors. The office floors (Fig. 2.5) are supported by long span (up to 14m) solid section Universal Beams acting compositely with a 130mm deep re-entrant concrete deck. Arup Fire designed an engineered fire protection layout, reducing fire protection to all primary members (beams and columns) from 2-hours to 90 minutes and leaving secondary beams unprotected. This was considered appropriate, because of the robust structural form that had deliberately been chosen by the structural engineer with structural fire engineering in mind.

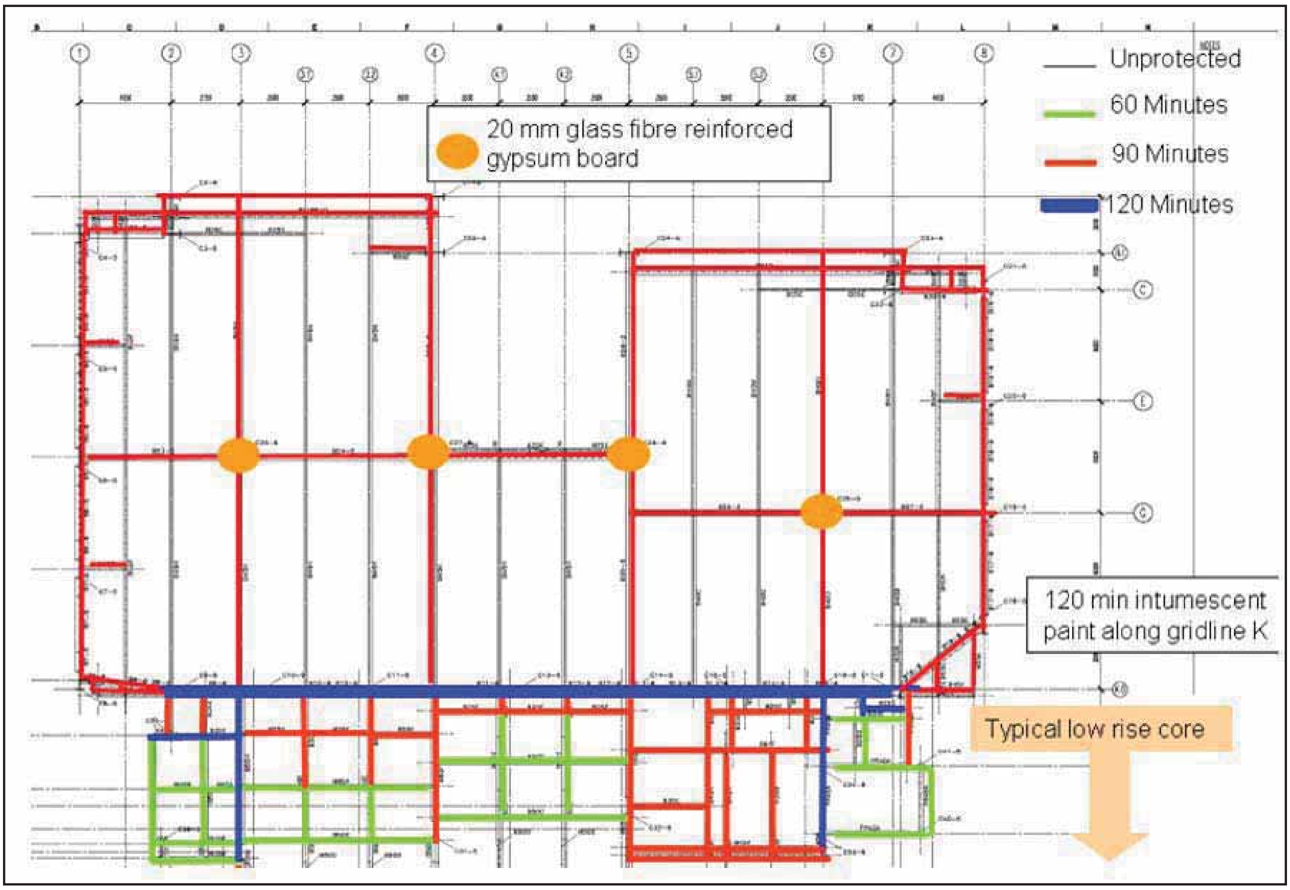



Fig. 2.5 Structural floor layout

To demonstrate that this would provide an adequate level of protection, a finite element analysis was carried out using the commercial modelling program ABAQUS. The first stage was to agree a reasonable design base fire scenario. The Parametric Fire in Eurocode 1 Part 1.2 (BS EN 1991-1-2 2002) was proposed with the fire located at a single level only. However, due to the atria penetrating the normal floor to floor compartmentation, it was agreed that two models would be run in order to fully evaluate the structural response: a single storey model with the onerous Parametric Fire and a multi-storey model with a less severe Parametric Fire than the single storey model. The models were then created giving a realistic representation of the structure including non-linear temperature dependant material properties, which are necessary to capture the kinds of large displacements seen in structures under fire load.

In the single storey model, with the more severe fire, maximum deflections (Fig. 2.6) over unprotected beams were approximately 2m (Span/7.2). By comparison, the Cardington test series (Newman *et al.* 2006) saw a maximum deflection ratio of approximately Span/10. The response of protected primary beams was much less extreme with maximum deflections of approximately 500mm (Span/20). The model demonstrated that stability and compartmentation were maintained. The multi-storey model indicated smaller beam deflections (approx. Span/10) due to the more reasonable fire. Even though columns were affected over a number of floors, there was no indication of column instability.

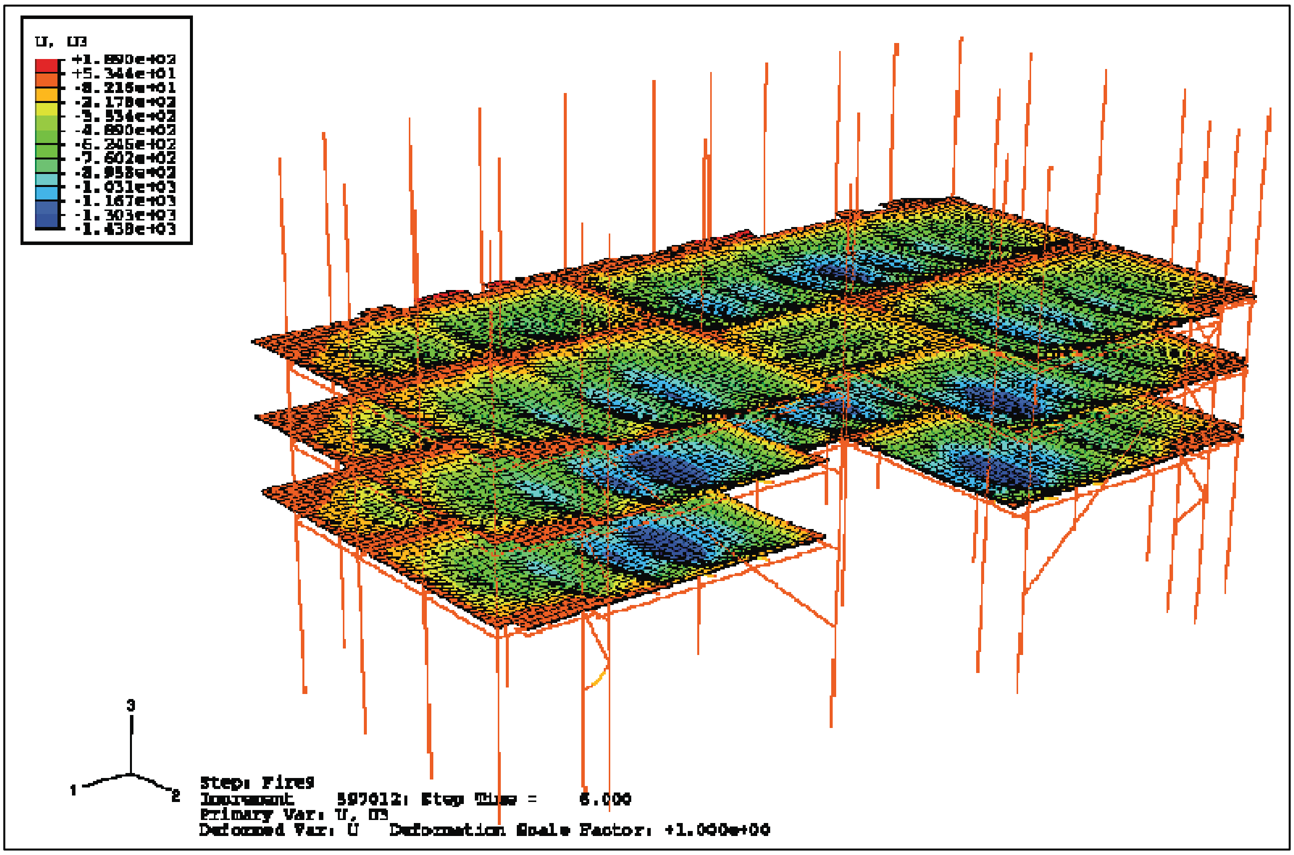



Fig.2.6 ABAQUS thermo-structural model of a full floor, showing deflection contours

A code-compliant (ADB (2006), PD 6688-1-2 2007) fire protection layout of the single floor model was also assessed and showed considerable structural movement. It is commonly assumed that a building designed to code requirements will be relatively unaffected by fire. This analysis demonstrated weaknesses in the structural design that would not normally be observed. The finite element analysis therefore allowed us to demonstrate the robust nature of the building, rather than assuming that code-compliant protection would be enough.

A close relationship was maintained with the approving authorities and their designated 3rd party checker throughout the modelling project in order to ensure that they were happy with the modelling approach and the validity of the approach. Approval was granted on 29th December 2006, achieving significant savings for the client, not only in terms of the cost and the consequential reduction in required future maintenance, but also the benefit to the project program and better architectural finishes to exposed elements. Additionally by reducing the amount of spray-on intumescent the environmental impact of the building and hazard to workers is reduced.

This is understood to be the first building in the UK that has been approved using a multi-storey fire analysis as a fundamental part of the approvals process and is now widely seen as a benchmark for structural fire engineering in London.

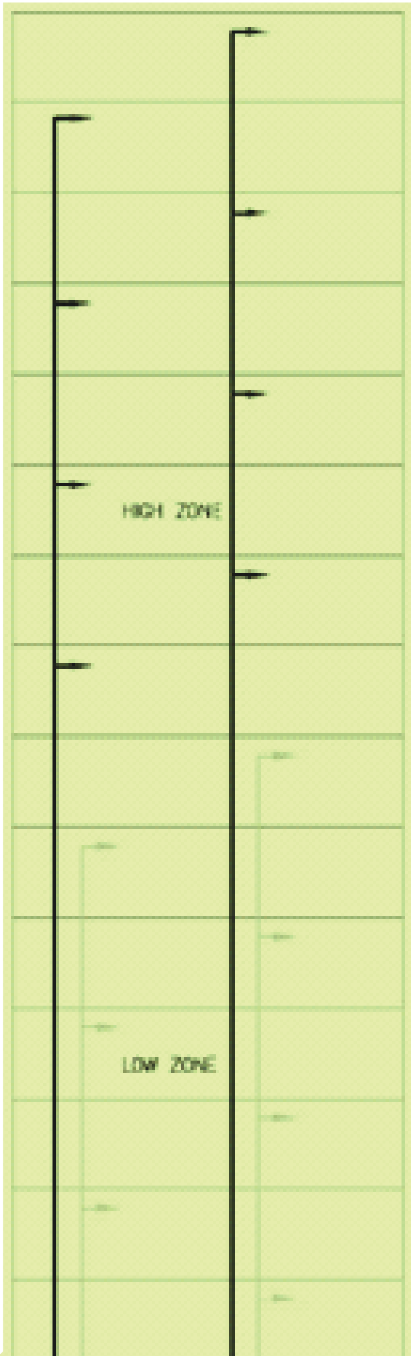
## 2.3 EXTREME EVENTS STUDY

Heron Tower originally started design in 1999 with the first planning application made in 2000. The building attracted controversy from the outset due to its proximity to St Paul's Cathedral. English Heritage pressed for a public inquiry, the outcome of which was decided by the then-Deputy Prime Minister John Prescott. The tower was finally given Planning Approval in July 2002. In the delay between the application being made and consent being given, the security situation in the world shifted due to the September 11th attacks on the World Trade Centre. Suddenly, fire and life safety in tall buildings was brought to the forefront of the world's attention.

A threat and risk assessment was carried out by Arup Security which identified a fire on multiple levels as a credible extreme event. To cope with this, Arup Fire designed the sprinkler system with a number of significant enhancements. Key to this was splitting the system into two separate sub-systems, with each sub-system being served by a separate rising main serving alternate floors, a separate tank with an infill from the town main to increase the capacity of the water supply and separate duty standby pumps.

A standard sprinkler system (BS EN 12845 2004 + A2 2009) would be designed to provide water flow through 18 heads for a period of approximately 1-hour. In the event of a fire on more than one floor, the water supply would be exhausted more quickly, possibly before the fire brigade had been able to access the building to fight the fire. The enhanced system will be able to provide water for at least 1-hou r if the fire is situated over two levels, and for longer than a standard system if the fire is situated over multiple levels.

The two separate risers have also been located on separate sides of the building thereby reducing the potential for an external attack on the building to completely knock out the sprinkler supply.



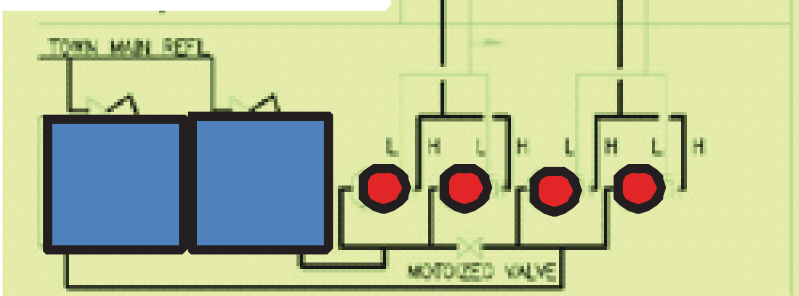



Fig. 2.7 Enhanced sprinkler system layout

Hence if one of the sprinkler rising mains is taken out of action, the second main should still remain in operation to supply every other floor. The benefits of providing an enhanced sprinkler system were seen throughout the design, with relaxations being given by the District Surveyor in a number of aspects relating to fire safety and also in the structural fire engineering design.

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# 3 ADIDAS LACES

Summary

The new adidas-headquarters, called 'Laces', has been opened recently in Herzogenaurach, Germany. The building consists of a 5-storey high ring of office modules, which are surrounding an atrium. For a slim appearance, structural members of the building have been left unprotected where possible and coated by a thin layer of intumescent painting where necessary. The fire resistance has been verified using methods of fire engineering.

Compartment temperatures have been calculated using the zone model CFAST. Input parameters such as fire load and compartment dimensions have been provided by the architect. As an important parameter, the opening area of the compartment has been varied in a parametric study to determine the relevant fire scenario. This fire has been superimposed with a local fire scenario.

The transient temperature fields inside structural members have been calculated using finite element software. Calculations were based on temperature dependent material properties for steel and intumescent coating.

The smoke exhaust of the atrium was designed using the CFD-simulation FDS.

## 3.1 BUILDING DESCRIPTION

The considered structure in this case study is the new representative headquarters of the sports-shoe-manufacturer adidas. The building, called 'Laces', has been opened in June 2011 in Herzogenaurach in Germany. It consists of a deformed ring of 5-floor high office modules that are surrounding a huge atrium. As there are two additional basement storeys below the office modules, the building all over consists of 7 storeys with a ground area of 61900 m², including the atrium. The 'Laces' offers workspace for about 1700 employees in offices, workshops and laboratories.

At the front side of the 'Laces', the ground floor and the 1$^{st}$ floor (further on referred to as storey 1 and 2) of the office modules have been left out to create a large open entrance to the atrium, as shown in Fig. 3.1. The different parts of the surrounding office building are linked by small bridges in every storey, leading through the atrium. Those bridges are called 'Laces', in analogy to a huge sports-shoe, and may be found in Fig. 3.1, as well.

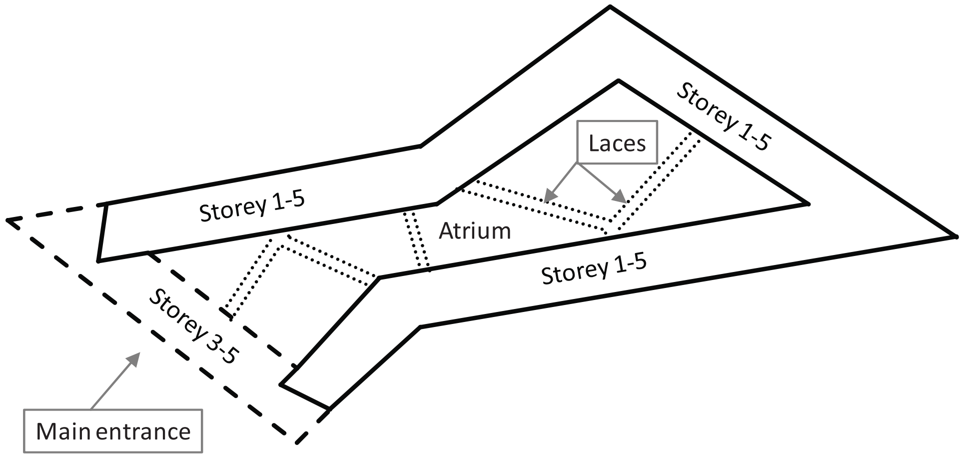


Fig. 3.1 Sketch of adidas-Laces

The main entrance to the atrium, which is spanned over by storey 3-5, is shown in Fig. 3.2 (left). Fig. 3.2 (right) shows an inside view of the atrium from the point, where the 'Lace' is connected to the storey directly above the main entrance. Because of the large dimensioning of the building in the whole building a sprinkler system and automatic fire detectors were provided.

Fig. 3.2 Main entrance (left) and inside view of atrium (right) of adidas-Laces (©adidas)

**3.2 APPLICATION OF STRUCTURAL FIRE ENGINEERING**

To span over the main entrance, storey 3 to 5 are supported by a construction of trussed girders with a height of three storeys and a length of 90 m. Additionally, secondary beams are included in this structure to connect each floor with the truss. In Fig. 3.3, the girder during construction phase can be seen. The location of trusses and secondary beams is also shown in Fig. 3.4.

Fig. 3.3 Truss girder spanning over main entrance during construction phase (©hhpberlin)

As the architect aimed at a slim appearance of the building, it was asked to leave the steel structure unprotected if possible and use thin layers of intumescent coating if necessary. A fire resistance time of 90 minutes had to be proved as an alternative to the normative requirement of an R90 protection.

Additionally, it was asked by the building authority to prove the smoke exhaust inside the atrium taking into account the 'Laces' leading through this compartment.

### 3.3 GENERAL ASSESSMENT STRATEGY

As it was allowed by the building authority to use methods of fire engineering, the concept for the truss girder was as follows. First the fire load was determined according to EN 1991-1-2 for office buildings. Using the $t^2$-method, the fire was simulated in a zone-model using the software CFAST. Additionally, the localised fire calculation according to EN 1991-1-2 Annex C was used to find the critical temperature. To be on the safe side the sprinklers are not considered for the structural fire safety design.

Finally, the compartment temperatures were used as thermal action in several thermal finite-element-simulations including steel cross sections and intumescent coatings to predict the steel temperatures. The load bearing capacity at t=90 min was calculated using the method of the critical temperature and where necessary using methods of simplified mechanical calculations, all according to EN 1993-1-2.

The smoke exhaust was proved using the CFD-model FDS.

### 3.4 FIRE SIMULATION

#### 3.4.1 Design fire

The investigated truss girder is located in storey 3 to 5 above the entrance. Fig. 3.4 shows the position of the truss girder and some of the secondary beams. As may be seen, the truss girders are crossing two different fire compartments, which are divided by the white coloured area, where the 'Lace' is connected to the storey. Thus the chosen fire scenario was a fire in one of these compartments.

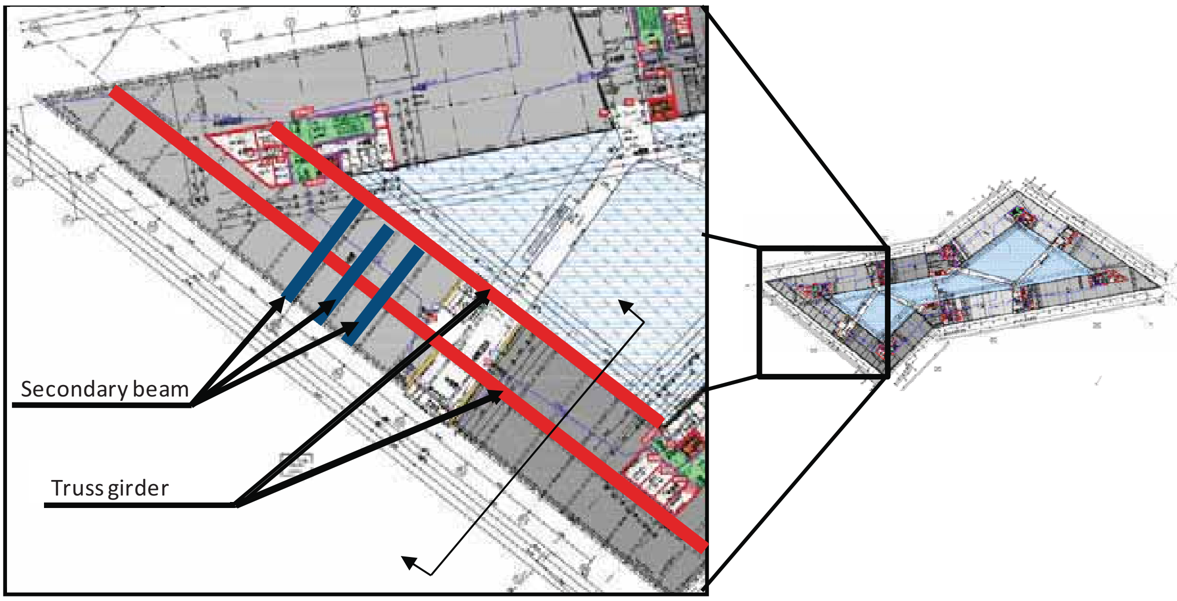


Fig. 3.4 Location of both trusses and some of the secondary beams

Both compartments are used as offices and it was confirmed by the client, that any future change in use will be declared and discussed with the building authority in advance. For this reason, it was possible to design the fire according to EN 1991-1-2. Thus the fire load was defined to 511 MJ/m², which is the 80%-quantile for fire loads in office areas. The rate of heat release was assumed to be 250 kW/m² and the fire growth rate, which was defined as medium, lead to a time constant ($t_{\alpha}$) of 300 s.
Using these input values, the fire was designed with the $t^2$-method. The rate of heat release can be seen in Fig. 3.5.

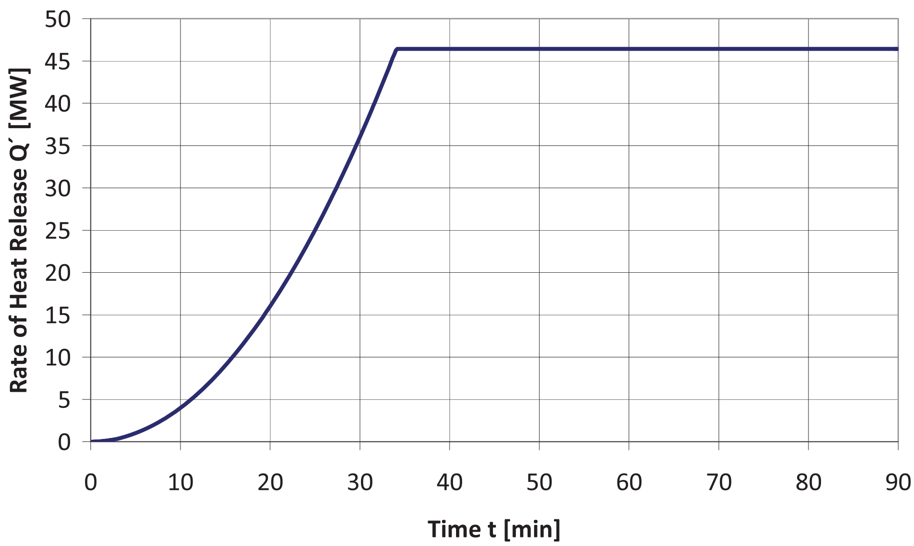


Fig. 3.5 Rate of heat release of design fire

### 3.4.2 Heat transfer analysis

The heat transfer analysis has been conducted combining two different models. First, the full compartment fire has been simulated using the two-zone-model-software CFAST. The geometrical approximation in CFAST consists of three connected rectangular compartments called C1_a, C1_b and C1_c, which are defined in Fig. 3.6. A visualization of the zone-model-compartments, including window areas and compartment connections (magenta), is shown in Fig. 3.7.

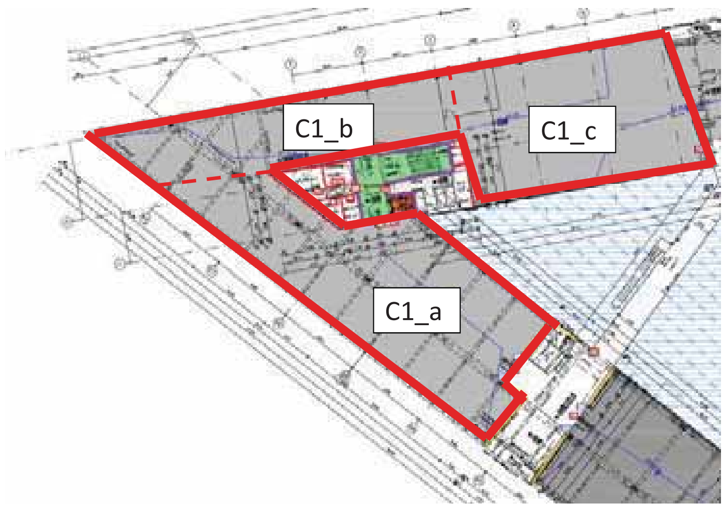


Fig. 3.6 Definition of compartments for multi-room zone-model-analysis

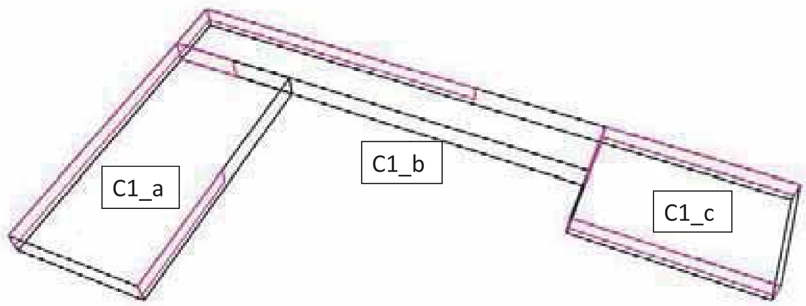


Fig. 3.7 Visualization of compartments in multi-room zone-model-analysis with CFAST

A critical parameter for the results of such calculations is the area of ventilation openings. As it is not possible to foresee, if and when a window is partially or fully opened or destroyed during a fire, the most critical opening area has to be defined. For the reason that it is also not possible to foresee if more ventilation openings increase or decrease compartment temperatures, a parametric study has been conducted. Fig. 3.8 shows the compartment temperatures using the minimum and maximum opening factor, which is defined as 25% and 90% of the whole window area.

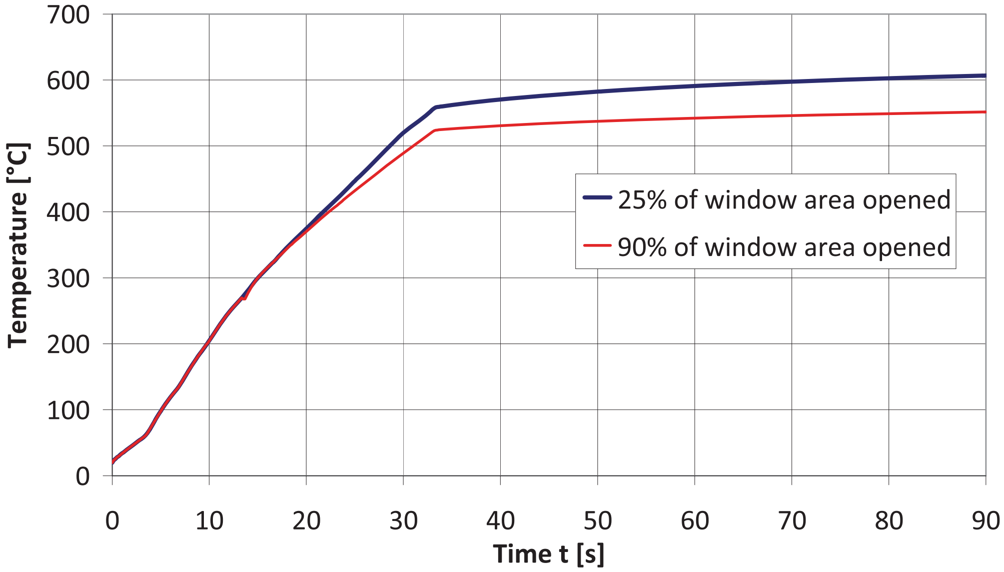


Fig. 3.8 Compartment temperatures with different opening factors

In addition to the zone-model-analysis for a fully engulfed compartment fire, a localised fire has been calculated using the same fire load density within a smaller area. For the calculation of this fire, the Heskestad-model according to EN 1993-1-2 has been used.

The resulting temperatures at the secondary beams and the diagonal braces of the truss girder are shown in Fig. 3.9.

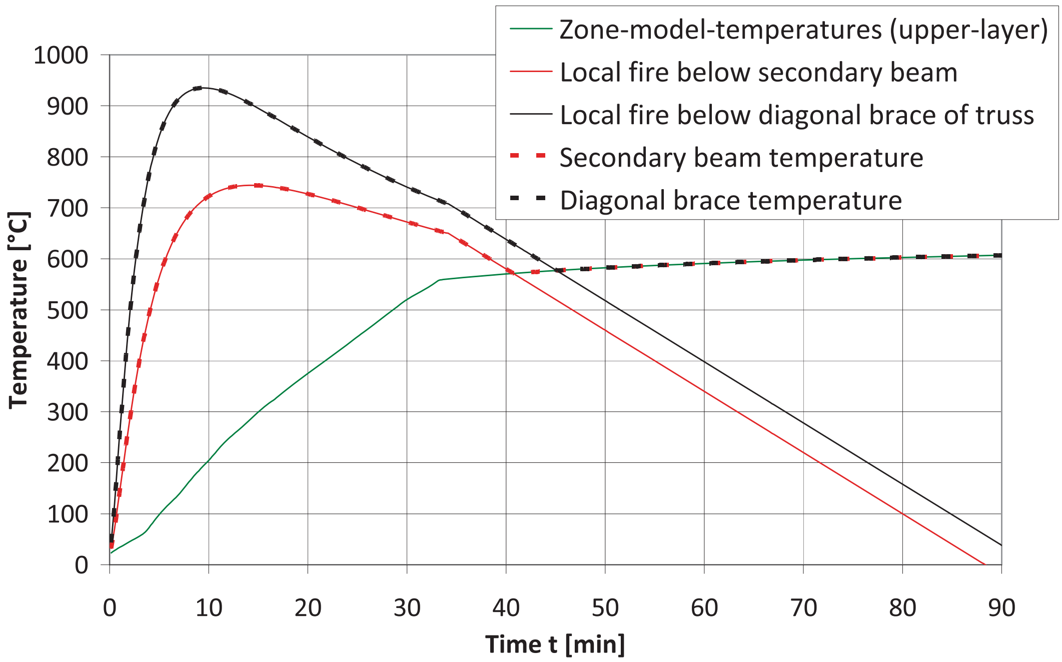


Fig. 3.9 Decisive temperatures for thermal analysis of structural members

It can be seen, that the temperatures calculated by the local fire model are decisive during the first 40 min in fire, while the temperatures calculated with CFAST are higher afterwards.

It has to be mentioned, that the shown curves for local fire temperatures have not been used for all parts of the thermal calculation of the structural members. When the flame height is reaching the different members, the thermal loading for them has to be calculated using the heat flux from fire to member, instead of calculating the air temperatures. This leads to a higher thermal loading and thus has been taken into account for the thermal calculation of the structural members. As it is not feasible to combine heat flux and gas temperatures in one diagram, this is not shown here.

### 3.5 THERMAL RESPONSE OF STRUCTURE

The structural temperatures have been calculated by hhpberlin using ANSYS. The double-check was conducted by the Institute for Steel Construction using BoFire. For both calculations, the same thermal material properties have been used. The material steel was implemented using thermal conductivity, heat capacity and density according to EN 1993-1-2. For the thermal simulation of the intumescent coating, material properties according to Dorn, 2003 have been used, as there are no normative regulations available. However, as the values have been proofed against experimental tests, they can be used in a particular range. In Fig. 3.10, the temperature dependent material properties are defined in relation to their values at room temperature (20°C).

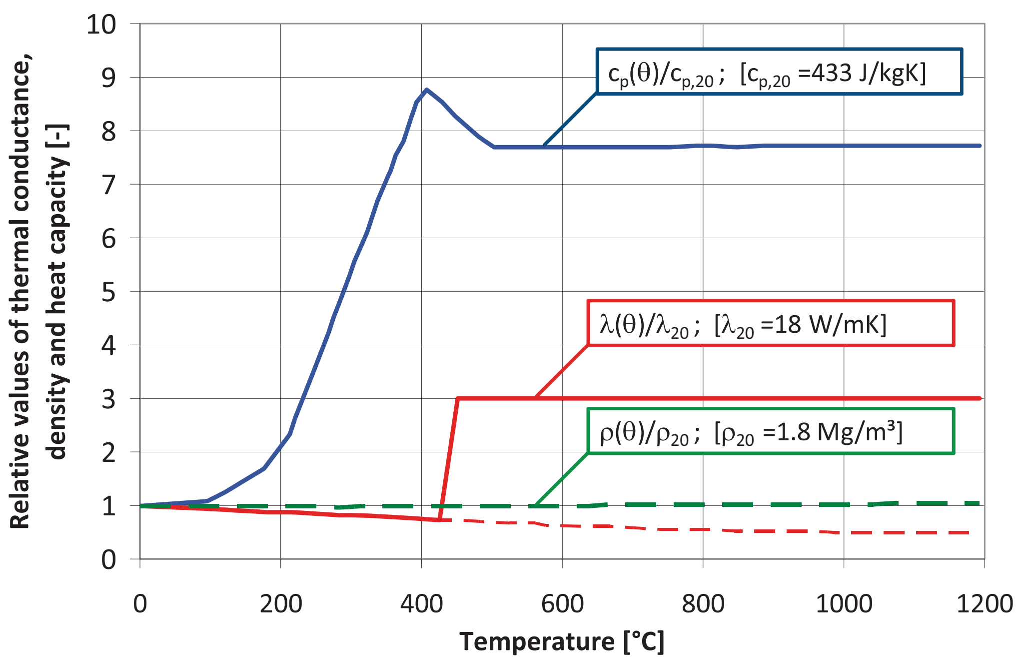


Fig. 3.10 Material properties for intumescent coatings according to Dorn, 2003

In Fig. 3.10 it can be seen, that the thermal conductivity $\lambda(\theta)$ is increasing at a temperature of 450°C. This steep increase has been manually implemented by Dorn, 2003 to cover the case of a local redemption of the intumescent coating.

The thermal response was calculated for the diagonal braces of the truss and for the secondary beams. The diagonal braces consist of two different circular hollow sections (20 and 70 mm wall thickness), while the cross section of the secondary beams is I-shaped with an additional middle flange. As can be seen in Fig. 3.11 for the example of the secondary beam, a two dimensional finite element model has been created based on the cross sectional geometry of the member, neglecting the middle flange on the safe side. In addition to the steel cross section, the intumescent coating has been modeled with a final thickness of 15 mm.

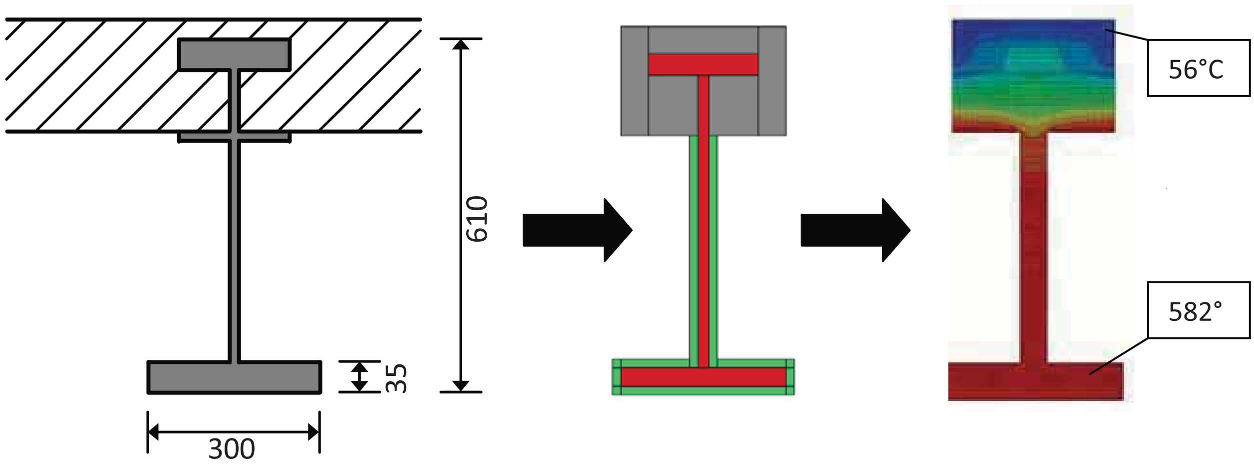


Fig. 3.11 Geometry of secondary beam, numerical model geometry and temperature field at t=90 min

The numerical results for the example of the secondary beam covered by intumescent coating are shown as temperature curves in Fig. 3.12. Additionally, two important time points have been included in the diagram. After 400 s the flame height reaches the location of the member. Thus the thermal loading changes from air temperature to a direct heat flux into the member. After 2070 s, the temperatures of the compartment fire are becoming higher compared to the local fire temperatures, as the local fire starts to decrease after consumption of all local fire loads. So from this point on, the thermal loading is based on the results of the zone-model.

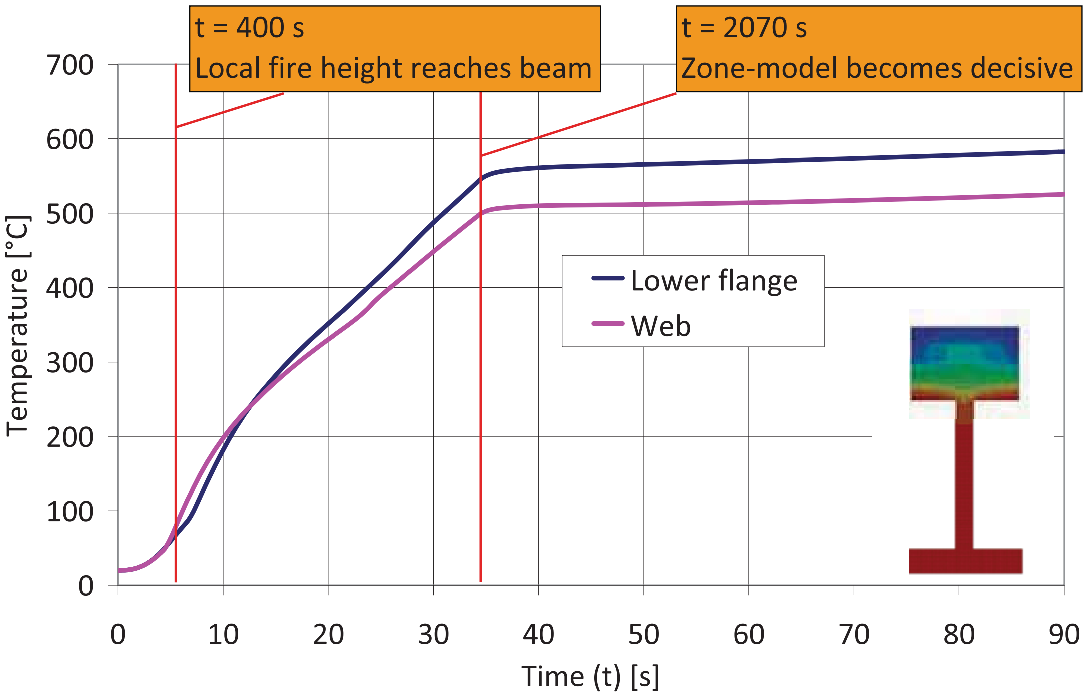


Fig. 3.12 Average temperatures of lower flange and web of secondary beam during design fire

### 3.6 MECHANICAL RESPONSE OF STRUCTURE

The mechanical response has been calculated using the critical temperature for the diagonal braces. As this method is not valid if stability problems may occur, this has been checked as well. The calculated temperatures in the secondary beams were slightly higher than the calculated critical temperature according to EN 1993-1-2 (section 4.2.4). So the reduction factors for steel according to Table 3.1 in EN 1993-1-2 were used to calculate the load capacity in fire. This capacity was compared to the maximum mechanical load in fire, which is reduced for the reason of the reduced partial safety factors and combination coefficients.

As a result of this calculation, it was proved, that a thin layer of intumescent coating (R30) was sufficient to protect the secondary beams and some of the diagonal braces. Other diagonal braces, built of circular hollow sections with a wall thickness of 70 mm were even allowed to be left unprotected.

### 3.7 DESCRIPTION OF THE APPOROVAL PROCESS FOR THE FIRE ENGINEERING APPROACH

The whole fire safety concept has been set up by the fire engineering company hhpberlin. The building control authority accepted the concept and allowed a deviation from the German standards. According to those, a fire resistance for the steel truss of 90 min in ISO-fire-curve (R90) would have been necessary. As a replacement for the R90-classification it was asked for 90 min resistance in a design fire. As the needed engineering methods are non-conventional, the authority forwarded the conducted fire resistance calculation to the Institute for Steel Construction to be double-checked by fire engineers.

### 3.8 SUMMARY AND CONCLUSIONS

The fire resistance of a truss girder and additional secondary beams inside the adidas-headquarter has been calculated using methods of fire engineering. The fire has been calculated using a two-zone-model and a standardised method to calculate local fire temperatures. The calculated air temperatures and partially the heat flux from local fire have been used as thermal load for a thermal finite element calculation to determine the steel temperatures. This finite element analysis included an intumescent coating, which was used to protect the steel parts. Finally, the calculated temperatures were used to determine the load capacity after 90 min in design fire.

It was proved that circular hollow sections with a wall thickness of 70 mm were able to be left without any fire protection. Thinner hollow sections and all secondary beams had to be protected with intumescent coating for fire resistance class R30. Summing up, because of the use of fire engineering methods, it was possible to keep the slim appearance of the construction instead of hiding it behind thick layers of plaster board.

Acknowledgements

The authors would like to thank adidas for the permission to publish this case study.

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General information on adidas-Laces: http://www.adidas-group.com/de/pressroom/archive/2011/10June2011.aspx

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## 4 THE PINNACLE, LONDON
## Arup Fire Ltd

Summary

The project is a 63-storey office building known as The Pinnacle, proposed to be built in the City of London. The building is designed by KPF architects. The building profile tapers linearly with height up to Level 44, where the floor plates then cut back sequentially, forming a spinal wrap profile. The building has a highly irregular floor plate and a beam layout which changes from floor to floor. Standing at 288m high, The Pinnacle will be one of the tallest buildings in the City of London. The building is scheduled to be completed in 2014.

Fig. 4.1 The Pinnacle as part of the City of London skyline - visualization

### 4.1 INTRODUCTION

Structural fire analyses were performed by Arup Fire to develop an engineered fire protection strategy for the structural steel members of the building and to assess the robustness of the building in a fire.

An engineered structural fire protection strategy featuring unprotected beams and reduced fire rating was proposed, rather than relying on the prescriptive guidance defined by Building Regulations. Non-linear finite element analyses were carried out using the ABAQUS program by the structural fire team in London. There were several challenges in undertaking the structural fire analysis due to the shape and structural form of the building.

- The organic shape of the floor plate meant that the beams had to be arranged in a highly irregular layout.
- The architects expressed their desire to have large, clear spans with minimum number of internal columns to provide flexibility for the building tenants.

- The architects also wanted the perimeter columns to have a circular cross-sectional profile. These perimeter columns formed part of the lateral load resisting system of the entire building.
- 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.

These architectural design requirements were expected to push the limits of stability of the floor system and the overall building in fire. However, the outcomes of the analyses demonstrated that these design requirements could still be realised by incorporating minor changes that would not impact on the architectural and structural designs.

**4.1.1 Building structure and its effects**

The building geometry of The Pinnacle was developed to suit the proposed structural form, featuring a perimeter-braced frame. The pattern of the braces and columns were an essential part of the unique character of the building. The diagonal braces, which were crucial for transferring the shear forces in the building to the foundations, had their layouts optimised by the structural design engineers to resist the worst-case wind condition.

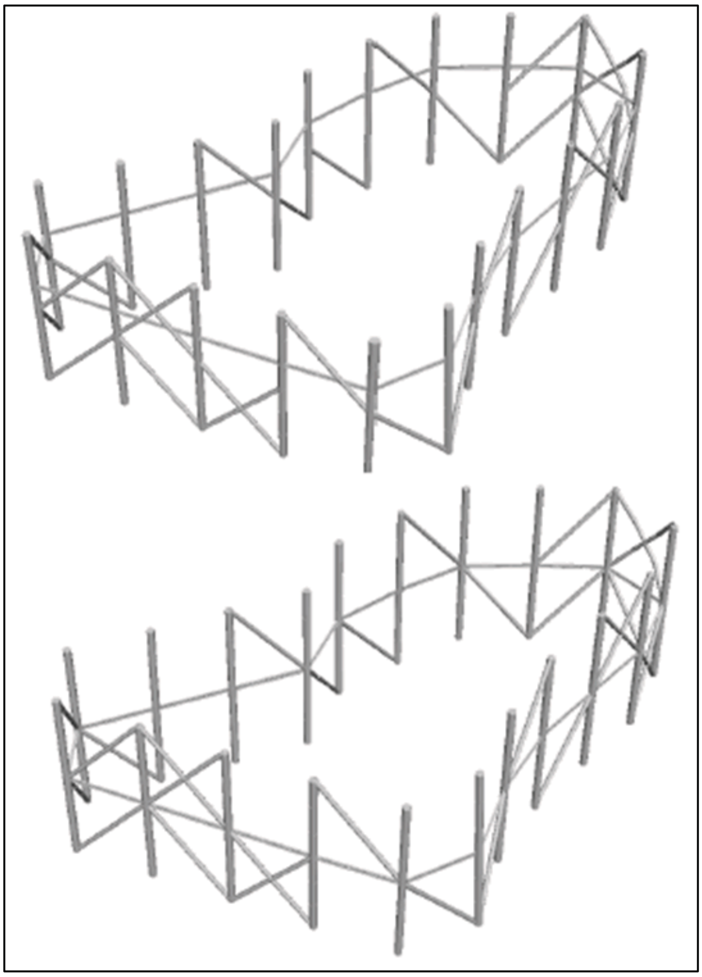

Fig. 4.2 Bracing arrangement details

To minimise the loss of lettable area cause by intrusion of the braces into office spaces, the braces have to change direction where they touch the intermediate levels between “mega-frame” 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. High-strength concrete (C80) was also used as infill for the perimeter circular hollow section (CHS) columns.

## 4.2 FIRE ENGINEERING

### 4.2.1 Structural fire engineering analysis

There were many unique aspects of this project which demonstrated innovation and creativity. These included:

- Very large complex models capturing localised and global behaviour
- A new methodology for modelling the external frame over multiple floors
- A modelling approach for a composite steel and concrete column system in fire
- Modelling techniques to allow simulations to perform efficiently
- An optimised fire protection layout tailored to the structure

Commonly in structural fire engineering, small and simplified representative portions of a floor within a building with a regular steel frame may be used to represent its overall response in fire. Usually, a building is assumed to be adequately restrained against sway by the lateral stability system, which is typically a reinforced concrete core, and assumed to be unaffected by fire. Because the lateral stability system of The Pinnacle is an optimised steel bracing system located around the entire building perimeter, the entire floor plate had to be modelled. The common assumption that the lateral stability system was not affected by fire could not be applied for this building. This is the first structural fire analysis where the lateral forces caused by wind, and the entire lateral stability system, were modelled at high temperatures.

The global behaviour of three separate extensive full-floor plates comprising irregular cellular beam arrangements were analysed. A novel approach taken in the analysis was the investigation into the behaviour of the building's outer lateral load supporting, diagonal grid structure, including mega-frames spanning three storeys and incorporating the effects of wind when exposed to fire. Detailed models of part of the floor plate were also analysed to capture complex and highly localised firerelated structural phenomena such as webpost buckling of the cellular beams. The models that were developed were not only the largest created for analysing structural fire performance within Arup, but they incorporated a very high level of detail and complexity to allow for an accurate dynamic representation of the structural response at high temperature. Without the use of such advanced methods, the proposed solution would simply not be possible given the sheer complexity of the structural arrangement.

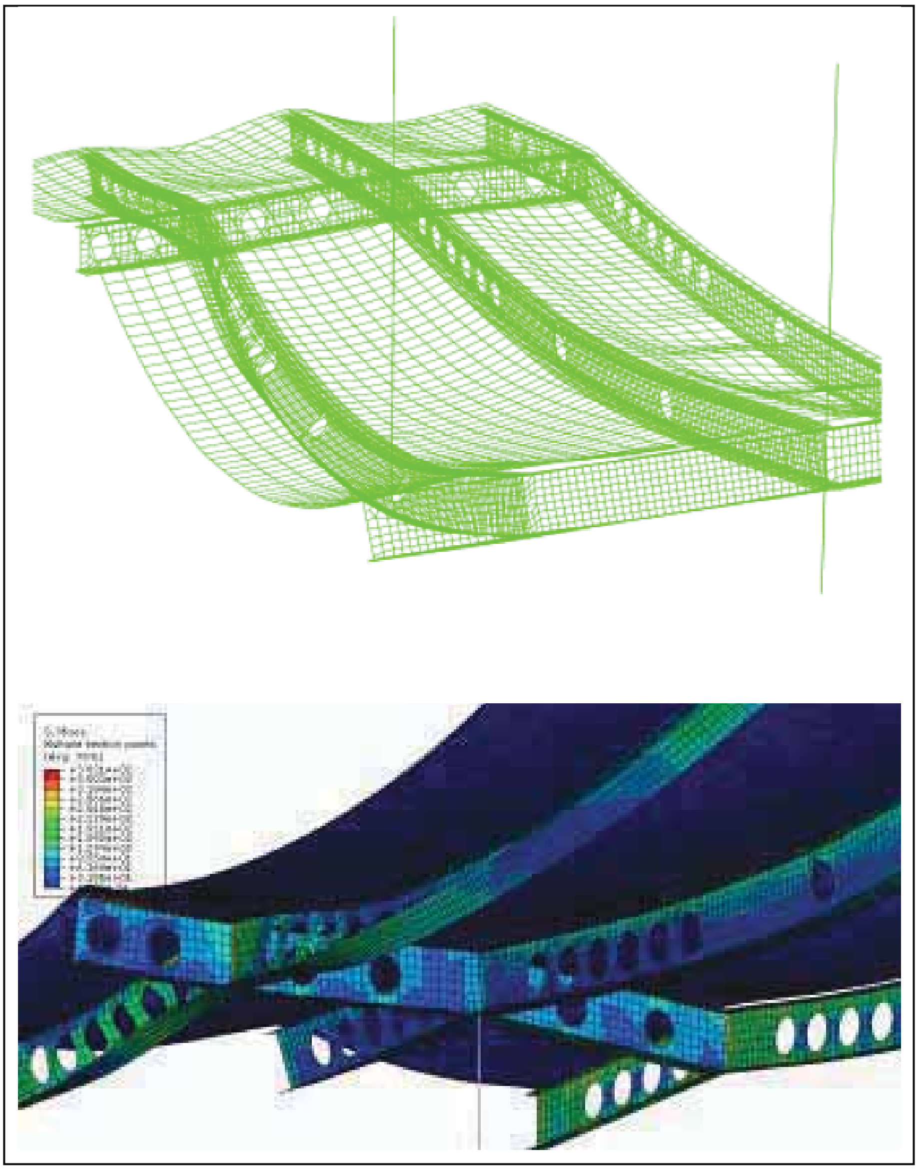

Fig. 4.3 ABAQUS models observing localised behaviour of composite floor employing cellular steel downstand beams

The analysis is the first of its kind to assess a multi-storey braced external tubular system (diagrid) that spans over 6 levels with mega-frame floors at every 3 levels. Wind effects and redistribution of forces that are transferred through this irregular system by membrane forces within the slab have been quantified. It is the first analysis in structural fire engineering which quantifies the heating and cooling phase over an entire 3,000m$^2$ floor-plate and its effects on connections and structural elements including the diagrid. The analysis incorporated beam and column connection capacities and partial shear composite action between the slab and beams.

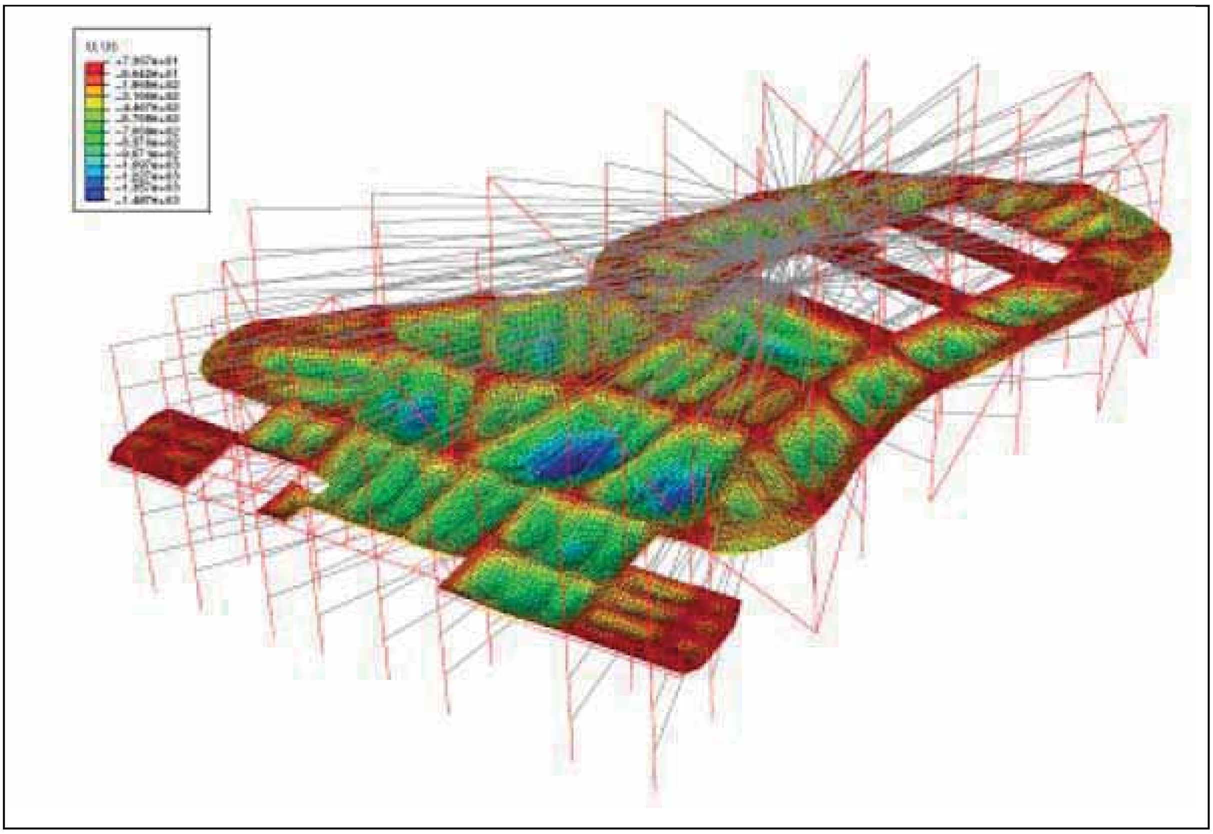

Fig. 4.4 ABAQUS model of one complete composite floor plate

A bespoke methodology was developed for this project to simulate the behaviour of composite columns comprising concrete filled steel tubes, when exposed to fire. The CHS columns filled with high strength concrete provided numerous benefits. As the steel tube gradually loses its strength in fire, the loads are transferred to the concrete infill. However, the steel tubes, although having lost its strength, would confine the concrete, preventing the concrete from spalling. The concrete acts as a heat sink and prevents localised buckling of the steel section at elevated temperatures. There was limited or no information available on the fire performance of many of the structural systems that would be used for the building. This was due to the sheer size of the structural members; for instance, the 1m diameter un-reinforced concrete perimeter columns and the long span cellular beams which had partial composite action with the supporting decks. Additional verification with finite element analysis against limited test fire test data had to be done specifically for the structural elements of this project. Factors of safety were then applied to the modelling by applying lower material strengths or by applying higher temperatures to the structural elements, than would be predicted by the thermal analysis. The outcome of this project was a fully quantified solution for the proposed building in great detail, incorporating connection forces, stresses, strains and deflections throughout allowing for an understanding of the strengths and weaknesses of structural design in terms of fire for both tall buildings in general and those specific to The Pinnacle.

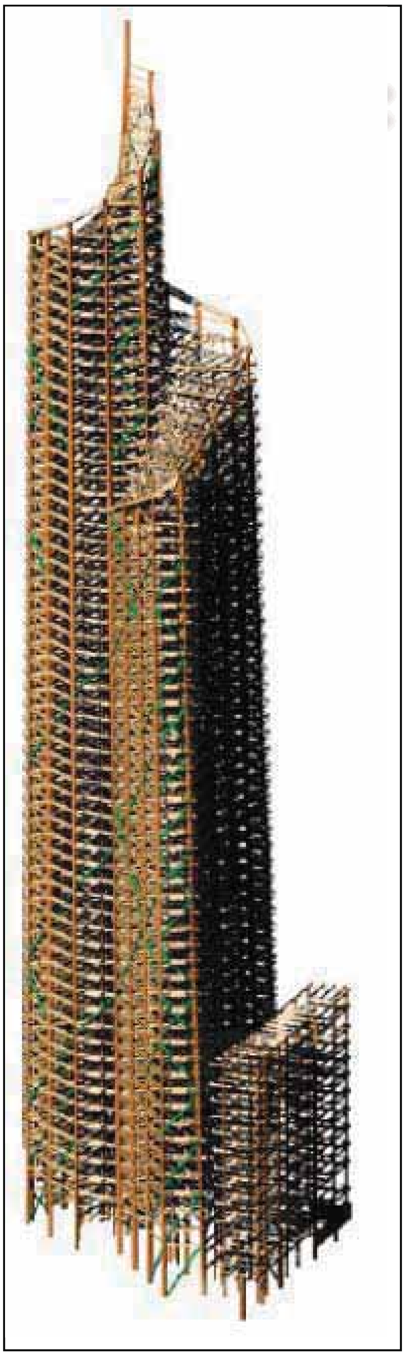

Fig. 4.5 Full-structure frame model

### 4.2.3 Value and benefits to design

The analysis resulted in an optimised fire protection arrangement, which is tailored to this complex structure, increasing its robustness and minimising fire protection material. The client benefited from a vast financial saving due to the reduction of fire protection material that was specified by Arup in comparison to what would have been required under prescriptive Building Regulations (2000) and their amplification documents (ADB 2006, LDSA 1997). The client also received a robust and quantified solution for their structure that allows them to inform and sell-on the value of the building with respect to its safety in future events. The occupiers of the building were provided with a structure designed to withstand specific extreme events, rather than unknown safety levels when designed to prescriptive code requirements. This project demonstrated significant value in undertaking detailed analyses to assess the structural fire robustness of unconventional and iconic buildings. It showed that a structure can be designed to perform well in fires when close design coordination is provided between the structural engineers and fire engineers.

The local community benefited from reduced damage to the environment through reduced use of noxious materials that can be common in fire protection. The type of fire protection for the steel members recommended by Arup Fire would be applied offsite. This would increase the efficiency of assembly of steel work on site and minimise the application on site which would pose significant occupational health and safety issues, such as working at height and overspray of fire protection. By optimising the amount of intumescent paint to be applied to the steelwork, this project has reduced the environmental impact as the structure becomes more environmentally efficient with regard to the volume of fire protection, had the fire rating been defined according to Building Regulations. The optimised fire protection allows the intumescent paint to be applied in a single coat rather than multiple coats of paint which needs additional curing time, creating wasted energy while the steel beams are cured in the workshop.

### 4.3 OTHER INFORMATION

The original question was to determine a cost saving with respect to the quantity of fire protection material required for the structure, as compared to prescriptive manufacturers' guidance (ASFP 2010). This became a complex study on unique structural forms at elevated temperatures, which required close coordination with the Corporation of London District Surveyors office, their third party checker Prof. Colin Bailey, the structural engineers, and the software providers ABAQUS: to create sets of modelling assumptions, and to apply advanced understanding to the complex behaviours observed as a result of fire. Through earlier research, close development work with the University of Edinburgh and a proven history of modelling tall, iconic structures in fire had allowed us to provide a service on this project that gave confidence to the client and approving authorities that a safe and cost effective solution could be achieved.

Arup Fire's strong connection with universities and leading role in developing cutting edge research in the field of structural fire engineering was a big advantage in helping to overcome some of the unique challenges that the project faced. A closely-knit working team of specialised structural fire engineers with a vast experience of numerical modelling techniques for this type of project was a feature that is unique to Arup. These factors result in a capacity to provide and complete such a challenging task. It is this capability which no other competitor can provide.

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# 5 FIRE PERFORMANCE OF AN OFFICE BUILDING WITH LONG-SPAN CELLULAR FLOOR BEAMS - BRITOMART EAST, AUCKLAND

Summary

This case study describes the application of a structural fire model to a 12-storey office building in Auckland, which was one of the first New Zealand projects to use long-span cellular floor beams. The structural fire model employed is known as the Slab Panel Method (SPM) and was developed by the Heavy Engineering Research Association (HERA). The SPM predictions of peak deflection under fire were investigated by more accurate Abaqus/Explicit simulations for a range of design fire severities and indicated that, for this form of construction, there is a tendency for the bottom flange of the cellular beams to displace laterally (which has recently been verified experimentally in the RFCS FiCEB+ project). From these analyses it was demonstrated that the passive fire protection could be eliminated from the long-span secondary beams, with only elements critical to the structural stability of the floor requiring the application of fire proofing materials. The resulting 80% reduction to the passive fire protection on the long-span cellular beams led to significant cost savings to the Client.

## 5.1 INTRODUCTION

The fire resistance of steel structures in office buildings, and their inelastic reserve of strength in fully developed fires, has received significant attention internationally. Through New Zealand's performance-based Building Code, there has been a strong focus on designing for the expected performance in fire rather than simply adopting traditional prescriptive requirements, which typically involve the application of passive fire protection to all structural steel members; this is especially the case in sprinkler protected buildings, given the very high effectiveness of sprinklers in preventing fire growth reaching full development (see Feeney and Buchanan). As a consequence of this, in sprinkler protected buildings the inelastic response in fully developed fires is an acceptable ultimate limit state response provided that collapse does not occur and the floors continue to function as effective fire separations.

One of the principal design procedures developed in New Zealand to take account of this inelastic reserve of strength is the Slab Panel Method (SPM). The SPM is the culmination of 8-years of research undertaken by HERA and the University of Canterbury, which extended Bailey's tensile membrane model into a design methodology for general application to steel framed buildings with steel-concrete composite floors. This paper presents the application of the SPM to a multi-storey office building together with the resulting performance-based design solution, which permits for partial fire proofing.

**5.2 GENERAL BUILDING DESCRIPTION**

Located in the Auckland City Central Business District, the 12-storey Britomart East Building provides 36,000 m² of office space over the Britomart underground train station. The ground floor includes street level retail and a large 10-storey atrium is built over a public pedestrian street, which passes through the centre of the building. The structural design solution was constrained by the location and load resistance of the existing concrete columns and piled foundations to the train station. A lightweight steel-frame using steel-concrete composite floors was selected, owing to the fact that it provided the maximum floor area whilst still ensuring that the foundations were not overloaded. An isometric view of the steel frame is presented in Fig. 5.1.

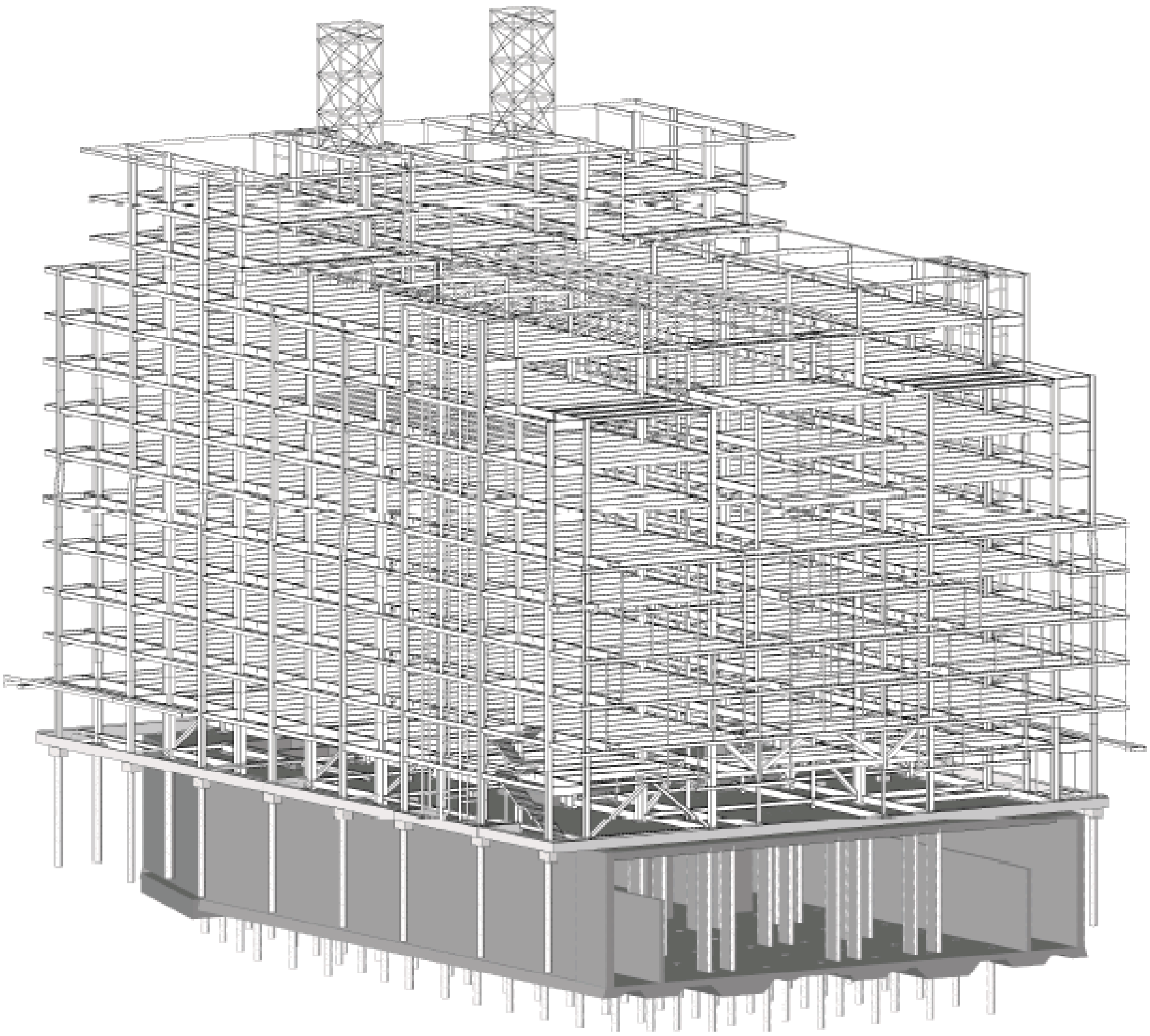

Fig. 5.1 Isometric view of Britomart East building

Due to geometric constraints imposed by the foundations to the existing train station, the structural grid for the building was not ideal for a steel-frame solution; moreover, the New Zealand seismic design requirements had a strong influence on the size of the structural members. As a result of these two influences, many of the structural steel elements within this project were sized for stiffness which resulted in a reserve in resistance under gravity loads at room temperature, thereby improving the performance in fire conditions. During the design development, two steel solutions were considered *viz.*: composite beams using conventional UB sections, and long-span cellular beams. The final solution that was selected was cellular beams, owing to the fact that they provide much greater flexibility for installation of building services, together with a lower steel weight per square metre.

The floor consists of a 130 mm deep concrete slab cast on ComFlor 60 profiled steel sheeting spanning 2.75 m between secondary beams. The secondary beams consisted of 496×171/190×56.1 kg/m Asymmetric Cellular Beam (ACB) sections with 300 mm diameter cells at 535 mm cross-centres. The ACB's spanned 12.0 m which, in turn, were supported by primary beams utilising 800×122 kg/m and 800×146 kg/m Welded Beam (WB) sections spanning 11.0 m between columns. Due to a span-to-depth ratio of 24, together with the fact that unpropped construction was used, the ACB's were supplied with a 40 mm pre-camber in order to satisfy total deflection requirements. A general arrangement of a typical floor is presented in Fig. 5.2. The lateral load resisting system consists of steel moment resisting frames. The external perimeter cladding is a mix of curtain wall glazing and concrete cladding panels.

With the exception of the roof level to the 10-storey atrium, the building is protected with an automatic sprinkler system in all areas. Passive fire separation is also provided between all floors with a 60 minutes fire resistance rating, as well as automatic smoke detection and a voice messaging system for staged evacuation of different parts of the building.

### 5.3 REGULARATORY REQUIREMENTS

The mandatory provisions for building work are contained within the New Zealand Building Code (NZBC), which consists of the First Schedule to the Building Regulations 1992. The NZBC is performance-based, which means that a designer has the freedom to use any method to comply, provided that they can demonstrate to the local building consent authority that the performance specified in the relevant Building Code clauses will be met. Structural stability during fire is a requirement of NZBC Clauses B1 Structure and Clause C4 Structural Stability During Fire.

The performance requirement in NZBC Clause B1 is that "Buildings … shall have a low probability of becoming unstable, losing equilibrium, or collapsing ... throughout their lives ... Account shall be taken of all physical conditions likely to affect the stability of buildings ... including self-weight, imposed gravity loads arising from use, … fire, ...". The functional requirement in NZBC Clause C4 is to "maintain structural stability during fire to: (a) Allow people adequate time to evacuate safely; (b) Allow fire service personnel adequate time to undertake rescue and firefighting operations; and (c) Avoid collapse and consequential damage to adjacent household units or other property".

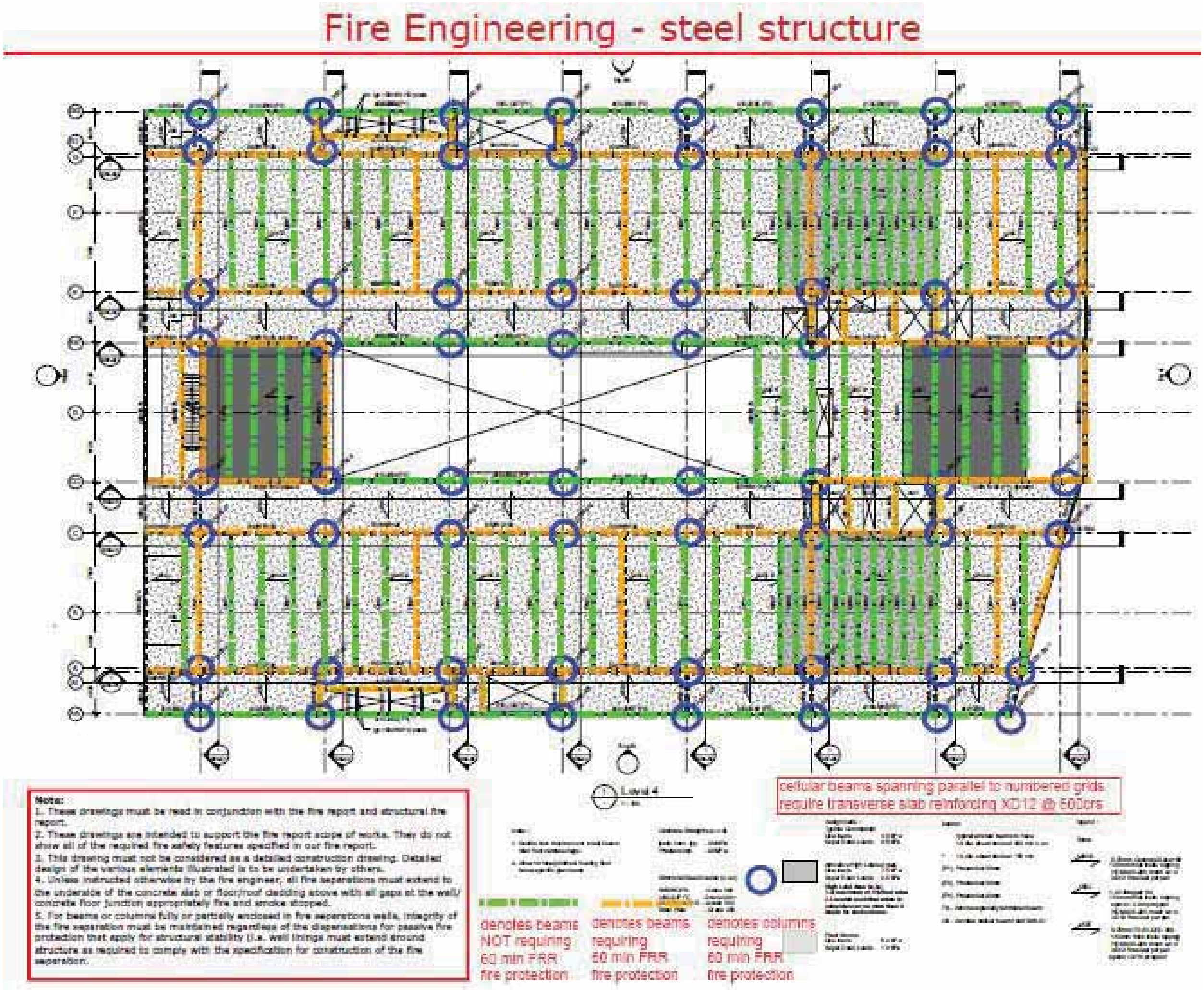


Fig. 5.2 General arrangement of typical floor showing passive fire protection to steel beams

Prescriptive compliance documents known as *Acceptable Solutions* and *Approved Verification Methods* (e.g. Codes of Practice) can be used to prescribe requirements for fire resistance ratings. A design that complies with the compliance documents is deemed to comply with the Building Code, but this is non-mandatory. A structural fire safety solution that is outside the scope of the compliance documents is categorised as an *Alternative Solution*. This type of solution cannot be rejected simply because it does not follow a prescriptive compliance document. The building owner is responsible for demonstrating how an alternative solution complies with the performance requirements of the Building Code. A common method is to establish that the alternative solution provides an equivalent level of performance.

For the Britomart East building, the acceptance criteria for adequate performance of the structure, as prescribed in the fire safety strategy report prepared for the building, is based on achieving the required level of fire resistance stated in the compliance document, which is deemed to comply with the Building Code. Fire resistance of the structure was verified using the HERA Slab Panel Method described below; this method has recently been acknowledged by the Authority Having Jurisdiction as an acceptable method for establishing performance of the structure during fire.

## 5.4 HERA SLAB PANEL METHOD

The Slab Panel Method (SPM) is used to assess which parts of the steel-frame require passive fire protection to maintain structural stability, whilst still achieving the performance requirements of the NZBC. The SPM is applicable to a wide range of design fire loads, providing design fire resistances between 30 to 240 minutes and is appropriate for most forms of concrete slabs that act compositely with the supporting steel beams. The methodology consists of dividing the floor into rectangular areas known as slab panels (or 'floor design zones'), with vertical support being maintained along the perimeter of each area through composite beams with applied passive fire protection; between these perimeters, unprotected composite beams are provided. It is assumed that the panels are subjected to a fully developed fire, resulting in the unprotected composite beams being subjected to very high temperatures and the floor area subjected to considerable inelastic demand. The extent of the inelastic demand is determined, and the available resistance in the fire situation at this point is incorporated within the procedure. The procedure also takes into account the temperatures that the unprotected steel beams can realistically reach in fully developed fires over large areas.

The method requires a design 'fire resistance rating' to be determined from an appropriate source; typically, this is the equivalent time of standard fire exposure $t_{ed}$ from EN 1991-1-2, Annex F. The fire emergency design vertical load for this time is then determined using the SPM procedure. The key differences between the SPM and the tensile membrane model developed by Bailey (which has been implemented within computer software such as TSLAB and FRACOF), are:

- SPM incorporates the contribution of the supporting beams directly into the flexural/tensile membrane slab panel load resistance (to enable the yield line pattern to be accurately determined), as opposed to TSLAB and FRACOF which only considers the contribution of the slab panel before the contribution of the supporting beams is added.
- As opposed to TSLAB and FRACOF, SPM implements a check for the vertical shear resistance of the slab panel.
- The methodology for determining the elevated temperatures of all components in SPM is more comprehensive and has been developed from an experimental programme by Lim.
- SPM allows for the slab panel supports to develop negative bending moment resistance, but does not take into account any lateral restraint of the slab panel edges.

- The limits on maximum deflection of the slab panel initially recommended by Bailey have been modified through experimental testing and analytical modelling undertaken in the NZ research programme.
- The SPM provides comprehensive structural detailing requirements to ensure that the floor panel can dependably develop the design deformations without loss of structural stability or integrity

As a further verification of the methodology, work conducted by the third author has also shown that the results from SPM agree favourably with those from the finite element program Vulcan.

**5.5 CONSIDERED DESIGN FIRE SCENARIOS**

Regardless of the very low probability that a fire in the sprinklered Britomart building would reach flashover conditions and adversely affect the structure (annual probability of less than $1.2\times10^{-5}$), the fire scenario selected to represent the design case is the low probability event of a fire not being controlled by the sprinkler system, which reaches full development. This assumes that the sprinklers do not operate and that a fire grows uncontrolled by any manual or automatic intervention.

A range of structural fire severities were determined and the SPM was applied to the most severe of these. The average structural fire severity (equivalent length of time of ISO 834 exposure) was 45 minutes, the maximum was 75 minutes and an 80% value was just under 60 minutes. As well as evaluating the response of the structure to a range of fire conditions, the post-fire cooling down period was also considered. For 45 minutes structural fire severity, the cooling down period was considerably longer at 255 minutes.

**5.6 FINITE ELEMENT ANALYSES**

The Britomart floor system consists of composite slabs supported by a network of primary and secondary steel beams (see Fig. 5.3). Under ambient temperature conditions, the floor is designed to act as a series of one-way load spanning elements. As specified in the SPM procedure, under severe fire the unprotected secondary beams lose their strength and the floor system responds as a two way ‘slab panel’ element. Each slab panel has 4 sides, with each side required to have sufficient strength to support the tributary loads direct from the slab panel. This means that for the sides of the panel supported on secondary beams, the supporting beams may need to resist a larger vertical load in the fire emergency condition than those present in ambient temperature conditions, even though the load per square metre is lower; this is because of the higher tributary area on these secondary support beams. For conventional solid web secondary beams, there is normally sufficient resistance as the beam size is governed by deflection or vibration considerations.

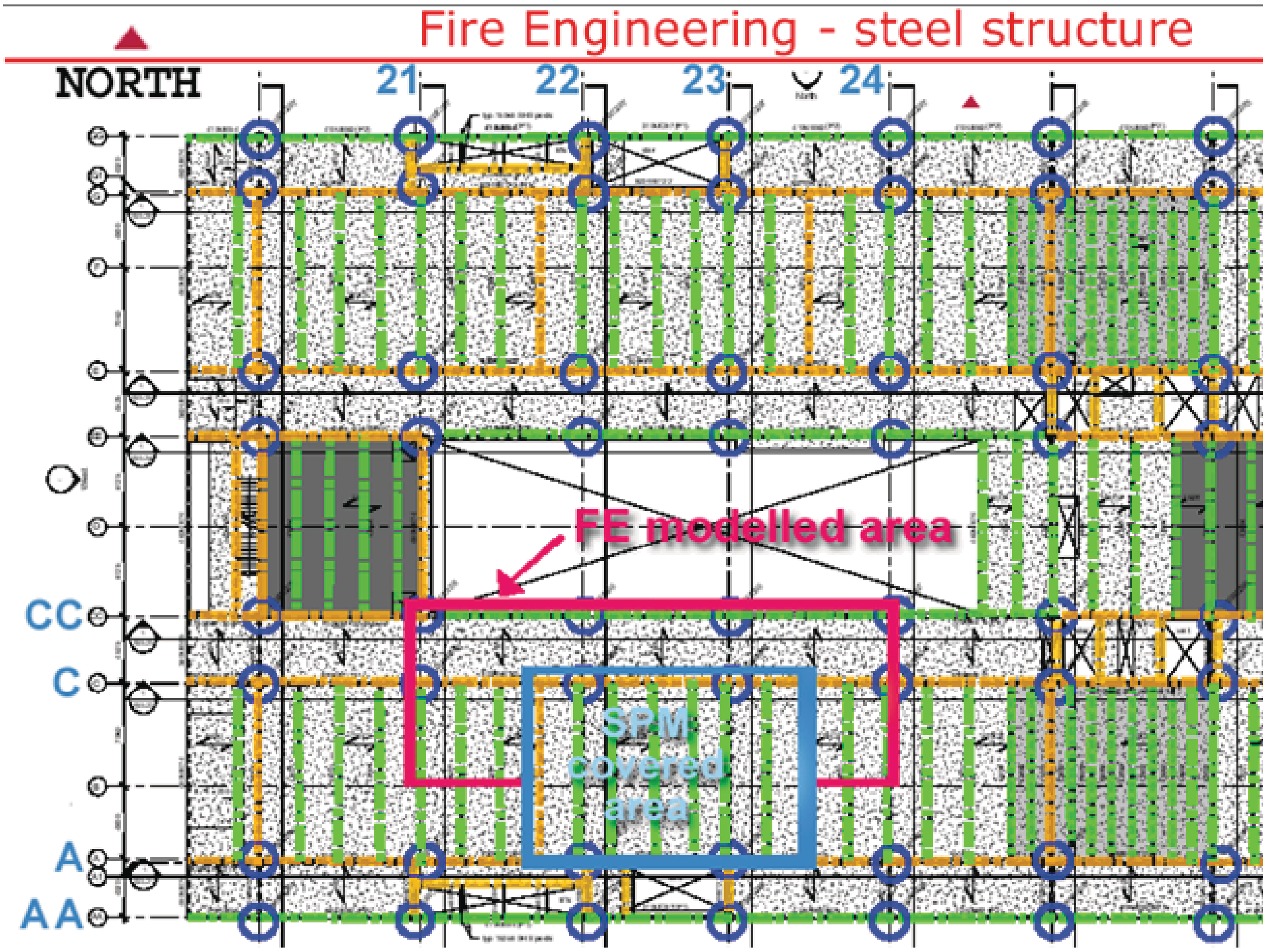


Fig. 5.3 Typical floor showing the fire engineering design and part of the building modelled in the finite element analyses

However, the secondary beams between grids A and C and grids E and G are Asymmetric Cellular Beams. These tailor-made beams are optimised for structural efficiency and there is insufficient reserve of strength in these to support the full fire emergency tributary loading from the slab panel; in contrast, the slab panel primary support beams have sufficient resistance to resist the full slab panel loading. To assess, *inter alia*, the validity of the SPM approach to the floor system, finite element analysis (FEA) of a representative portion of the typical structural floor system was undertaken. This type or analysis is not routinely carried out for fire engineering design but was required in this instance due to the modification of the strict application of the SPM.

#### 5.6.1 Software

The FEA was undertaken using ABAQUS/CAE/Explicit version 6.7-4 and performed in an explicit "quasi-static FE procedure" with temperature dependent material properties, as described below. The difficulties in performing successful highly non-linear analyses of concrete structures with temperature dependent material properties are well known. Implicit codes (such as SAFFIR, ABAQUS/Standard) do not always provide a convergent solution and FE simulations like those presented are almost impossible to perform with them. The Britomart finite element model size, complexity and the need for an extended duration deflection history dictated the explicit approach. This allowed the models to progress beyond failure in some of their regions, so large deflections could be captured in many simulations.

The first aim of the FEA was to determine the adequacy of the modified application of the SPM as part of the design solution. The second aim, which was equally important, was to determine the likely response of the structure to the range of fire conditions expected and at the end of the post-fire cooling down phase. For the first case considered, which was based on applying the prescriptive solution given in the Approved Document for Fire Safety (C/AS1), the analysis required simulation of the structural behaviour for approximately 45 minutes heating up and for the much longer 255 minutes cooling down period. The simulation of this long lasting fire condition is challenging in explicit codes, therefore "time scaling" and mass scaling were used to obtain the solution within a reasonable time frame (up to one day). A more detailed description of this analysis is given by Mago *et al*.

**5.6.2 Material Properties**

Both primary and secondary beams used grade 300 steel supplied according to AS/NZS 3679. The concrete had a characteristic compressive cylinder strength of 30 MPa. The temperature dependent material properties using EN 1994-1-2 were taken into account within the finite element models.

**5.6.3 Slab modelling**

The composite slabs used in the building consist of a 130 mm deep concrete slab cast on ComFlor 60 profiled steel sheeting. Full shear connection between the beams and the composite slabs was assumed. An equivalent reinforced concrete slab of 100mm thickness was used to represent the composite slab. This approximation has been previously shown to be valid in the modeling of experimental testing undertaken as part of the SPM development, provided that the reinforcement position and area is adjusted to give equivalent load carrying capacity.

**5.6.4 Connections**

All beams in moment frames were fully welded to the columns, while the webs to the cellular beams were bolted to the web of the primary beam or column with a web cleat.

**5.6.5 Boundary Conditions**

Columns were represented as extending to floor levels below and above the compartment in the FE model. At the lower level the columns were fixed or pinned as appropriate, whilst at the upper level they were axially loaded with design forces from the levels above. The boundary conditions allowed the columns to extend only upwards.

Fig. 5.3 shows part of the building on which FEA was undertaken. Lateral support conditions were varied from free to restrained (symmetrical boundary conditions) along grid lines 21 and 24, since finite element sub-modelling was not applicable in this case. In practice, all the slab panels are laterally restrained to some extent (which was incorporated into the FEA), whilst the SPM assumes no lateral restraint to any panel in the plane of the slab. Symmetry was assumed in the midpoint between grid lines A and C (see Fig.5.3).

### 5.6.6 Loading

At ambient temperature, the design imposed load in the office areas was 3.5 kPa. In checking the strength and stability of the structure at the fire limit state, the loads should be multiplied by the relevant load factors, which resulted in a fire emergency load of 1.9 kPa. The superimposed dead load on the floor was 0.5 kPa. In the first step of the analysis, the uniformly distributed loads (including the self-weight, superimposed dead load and fire emergency load) and column forces were applied in a smooth quasi-static explicit procedure. This step was followed by the fire loading step.

## 5.7 RESULTS FROM FINITE ELEMENT ANALYSES

Two cases were analysed in the investigation. The first case was based on applying the prescriptive solution given in the Approved Document for Fire Safety (C/AS1) involving application of passive fire protection to all steel members to achieve a fire resistance rating of 45 minutes. This was analysed for the natural fire condition conditions followed by a cooling down period so that the results of the SPM design solution and the Acceptable Solution could be compared. The deformed shape for the natural fire condition is shown in Fig. 5.4, which considers the area bounded by grid-lines CC/21-24 extending to the midpoint of the floor between grid-lines A and C.

The second case (which was implemented in the final design), is a more cost effective solution based on selective fire protection of the cellular beams comprising slab panel supports in the North-South direction, whilst leaving unprotected the cellular beams within the slab panel region. The edge beams were also left unprotected. Whilst the slab panel between grid-lines C and CC (also A and AA and the other side of the building) is satisfactory, it was found that the strain demands and deflections of the primary beams on grid-line CC were too high if these were left unprotected and laterally unstiffened.

From a preliminary FEA, it was found that twisting of the secondary beams in the positive moment region occurred during the heating stage, which led to significant lateral instability and movement of the bottom flange to the asymmetric cellular beams (this has recently been observed in the full-scale fire test conducted at the University of Ulster in the RFCS FiCEB+ project). To remedy this situation, as well as providing passive fire protection, transverse web stiffeners linking the top and bottom flanges were introduced at quarter points to the secondary beams forming the slab panel supports. A similar failure mode has also been observed in FEA of a floor using asymmetric cellular beams by Flint and Lane. In this case, the lateral instability was eliminated by providing a wider bottom flange than originally specified.

Fig. 5.5 shows the deformed shape at the end of the cooling down period of the natural fire condition at 300 minutes. As can be seen from this figure, most of the cellular beams were left unprotected, with the reduced deflections from the protected secondary beams that form the slab panel support clearly evident.

The key results from the finite element analysis were that the slab panel solution between grids A and C and E and G, which involved the cellular slab panel edge secondary support beams, was satisfactory with all deflections and strains within acceptable limits. Residual deflections at the centre of the slab panel after the cooling down period are approximately 800 mm for the structure with partial fire protection (span/15 cf. with the limit of span/20 limit given in BS 476-21 for the standard fire test), and 100 mm for the structure with full passive fire protection. These findings are presented graphically in Fig. 5.6.

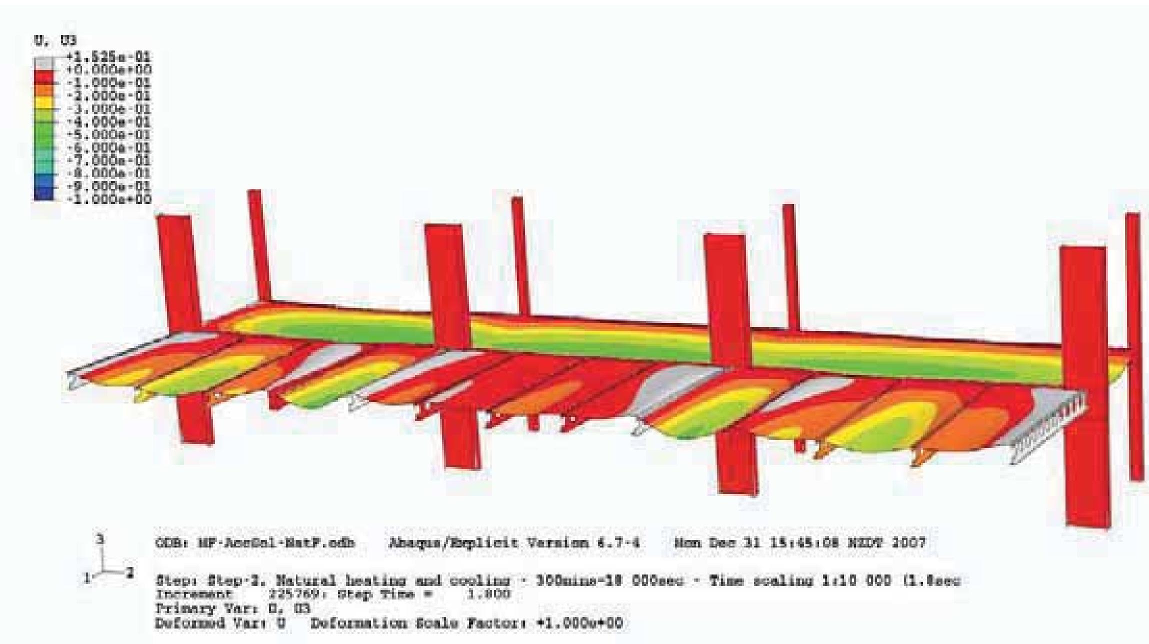


Fig. 5.4 Prescriptive solution with full fire protection based on Compliance Document C/AS1

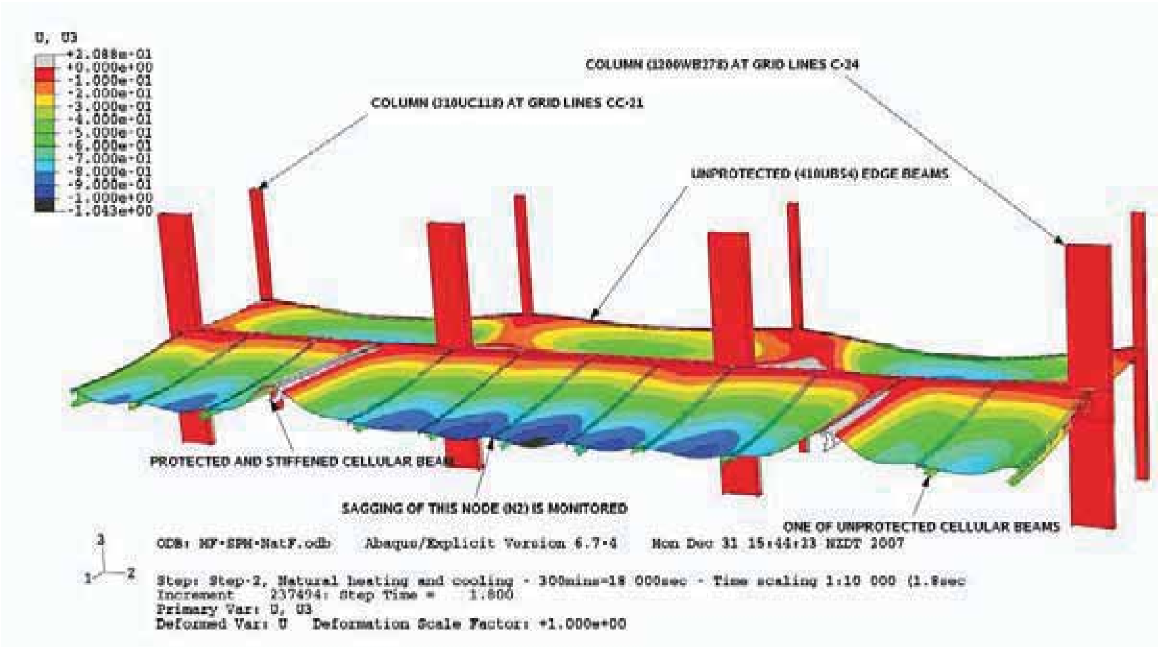


Fig. 5.5 Final design solution based on SPM analysis with partial fire protection

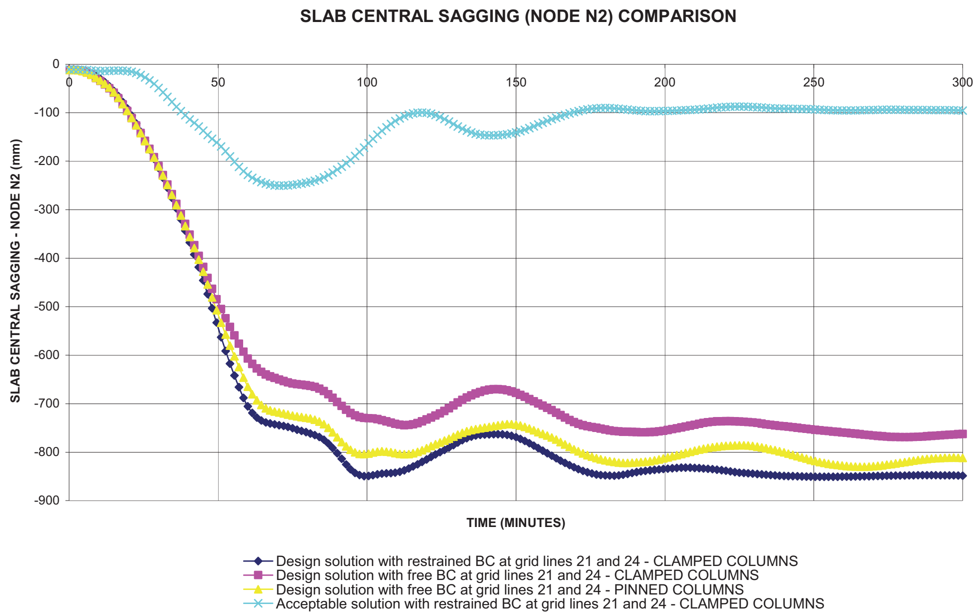


Fig. 5.6 Vertical deflection time history at the centre of slab panel for: partial fire protection (Design solution); and full fire protection (Acceptable Solution) based on based on Compliance Document C/AS1

As can be seen from Fig. 5.6, the deflections and strains in the all beams for the full fire protected solution are much lower than in the partial fire protection design solution. However, the post fire residual deflections would still be too large to reinstate the floor without requiring significant remedial work and either *in situ* beam re-straightening or replacement. Although, in theory, it might be possible to re-level the top of concrete with a levelling compound, the structure is unlikely to have the capacity to support the additional weight, particularly if more than one floor needed to be reinstated.

## 5.8 SUMMARY OF FINAL DESIGN

All structural columns have passive fire protection (60 minute fire rating) to ensure that the slab panel gravity loads are supported. The beams in the main lateral load resisting frames (grid lines A1, C, E and G) are all fire protected. Selected beams on the perimeter (grid lines AA, CC, EE and GG) are also passively protected. However, approximately 80% of the secondary beams do not require passive fire protection. This reduced extent of passive fire protection has resulted in a saving of more than NZ$300,000 for the project.

For the two cases considered in the FEA, the outcome was the same for all practical purposes. The large deflections and corresponding damage to the structure exposed to the effects of a severe uncontrolled fire would require replacement of the affected beams and floor slab, regardless of whether partial of full passive fire protection is provided. In both cases structural collapse is avoided, the load carrying capacity is maintained and the floors would be expected to function as an effective fire separation for the duration of the fire.

Passive protection of all beams does not eliminate the need for detailed assessment and probable repair of the floor after being subjected to fully developed fire. A much more effective fire safety strategy is to rely on the high effectiveness of the sprinkler system (Bennetts *et al.*), to suppress full fire development and to mobilise the inelastic reserve of strength from the floor in the very remote event of sprinkler failure. That is the approach behind the SPM method and the approach taken in this design.

## 5.9 REGULATORY APPROVAL

The Building Consent Authority responsible for regulatory approval (confirming that a proposed design complies with the Building Code), is Auckland City Environments. Review of structural design required specialist expertise beyond that available from Auckland City staff, so reliance was placed on external peer review. The structural fire design for the Britomart East building was reviewed independently for Building Code compliance on behalf of the Building Consent Authority.

## 5.10 CONCLUSIONS

This paper presents the application of the Slab Panel Method to a new 12 level office building in Auckland, New Zealand. From this case study it can be seen that design methods are maturing to a level where a dependable and robust performance can be predicted using the Slab Panel Method (SPM). The SPM analysis shows that if sprinklers fail to control a fire such that flashover is prevented, the structure retains sufficient strength to support design loads for the fire load condition. Accordingly, those parts of the structure which require applied fire protection can be specified with enough protection to maintain structural stability, and those parts which do not need this passive fire protection can be safely constructed without fire protection. The extent of fire proofing that is not required to satisfy Building Code performance criteria has been identified, resulting in a significant cost saving to the project.

Validation of the dependable structure performance with the reduced level of fire protection was made using finite element analyses. The finite element analyses also show that, for a structure protected with passive fire protection (as required to comply with a prescriptive fire safety solution and exposed to the fire severity of a fully developed fire), large deformations can still be expected thereby requiring post-fire replacement of affected structure. This case study demonstrates that the SPM method can be used to assess structure performance in fire in a performance-based regulatory environment.

Acknowledgements

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# 6 KINGDOM STREET, LONDON
## Ramboll SAFE Ltd

Summary

This case study describes a UK project in which performance-based structural fire engineering analyses were conducted for an office building to be constructed in central London. The case study demonstrates how structural fire engineering, using both simple and advanced forms of analysis, provides value in a design framework. Design options, including the code-compliant design and alternative designs which can bring cost benefits to the owner, are proposed and described.

Fig. 6.1 The 4 Kingdom Street office complex; Architect's view

### 6.1 INTRODUCTION

The Cardington full-scale frame fire tests (Newman *et al.*, 2006) demonstrated the robustness and stability of composite-floor framing systems in the event of a fire, even when steel downstand beams are unprotected. A design approach based on the enhanced resistance of slab panels at high deflections due to tensile membrane action emerged from these tests, giving the prospect of eliminating fire protection safely from large numbers of steel beams. The approach assumed that protected edge beams surrounding a slab panel maintain absolute vertical support of the slab, with the intermediate secondary beams within the panel being unprotected.

The fire engineering design described here was carried out for a new 12-storey office building to be constructed at 4 Kingdom Street, in central London. It shows how structural fire engineering methods can be applied on a typical office floor level in order to optimize the inherent fire resistance of the structure and its fire protection schemes, to offer robust but cost-effective design solutions which achieve the required fire resistance. The finite element software *Vulcan* (2005), developed at the University of Sheffield, was used to model and analyse the 3D composite floor slab for a typical office floor plan.

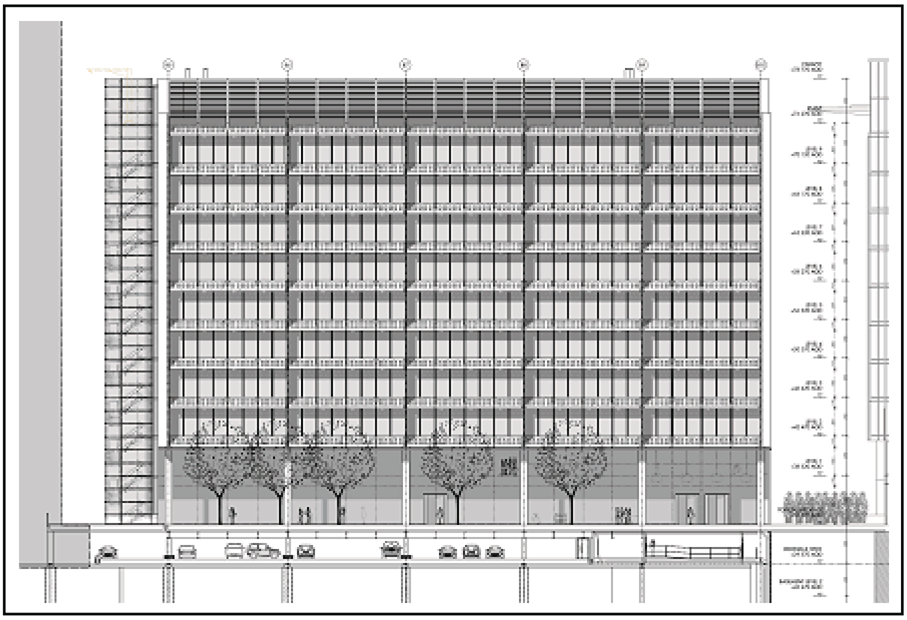

Fig. 6.2 Elevation of the 12-storey 4 Kingdom Street development

### 6.1.1 Building description

The proposed development is a 12-storey commercial building consisting of one basement level, a deck level, a podium (ground floor) level, and nine levels above, with an open-space plant area on the roof. For its above-ground office levels the development is a composite steel-framed and concrete floor slab construction, using long-span composite beams with web openings and steel decking. The floor plate measures 60m in length, and has a depth which varies from 32.25m to 20.25m (Fig. 6.3). The topmost populated floor of the building is approximately 36.6m above ground. The building has a concrete core, sited fairly centrally in the building, containing services and fire escape stairs.

This office building incorporates a high proportion of glazed or non-fire-rated elements on its façades, allowing for ventilation through sections of façade penetrated by fire. The exterior of the floor is surrounded by unprotected façade. It is assumed that there is a possibility that part of the façade will fail during fire; therefore, an alternative level of fire safety can be achieved by conducting a performance-based assessment based on a range of ventilation conditions.

## 6.2 FIRE ENGINEERING

### 6.2.1 Structural fire resistance strategy

For office buildings over 30m, Approved Document B (ADB 2006) of the Building Regulations recommends a fire resistance rating of 120 minutes (R120) to structural elements. However, the Building Regulations (2000) essentially permits the use of a performance-based fire engineered approach to achieve an alternative level of safety, instead of the prescriptive guidance in ADB (2006).

An engineered analysis has been undertaken to determine the effect of fire on the building structure in order to determine an efficient fire protection scheme to comply with the requirements of Part B3 of the

Building Regulations, which states that 'The building shall be designed and constructed so that, in the event of fire, its stability will be maintained for a reasonable period'. The assessment was first based on the "equivalent time of fire exposure" method described in the published document PD 6688-1-2 (2007), to assess structural performance in fire against actual compartment conditions. In this context analyses were conducted at various levels; code-compliant isolated member selection, the BRE-Bailey Method (Bailey 2000a, 2000b) for slab panels with unprotected steel beams within protected edge-beams, and Finite Element Analysis (FEA) to evaluate the integrated structural response of large subframes of the composite structure in natural fire scenarios when passive fire protection is eliminated from most of the secondary beams. This performance-based design approach was able to show that a reduced period of fire resistance, compared to the prescribed values, was sufficient to meet the functional requirement of the Building Regulations, and that reduced fire protection could offer significant cost savings to the client while maintaining the required safety levels

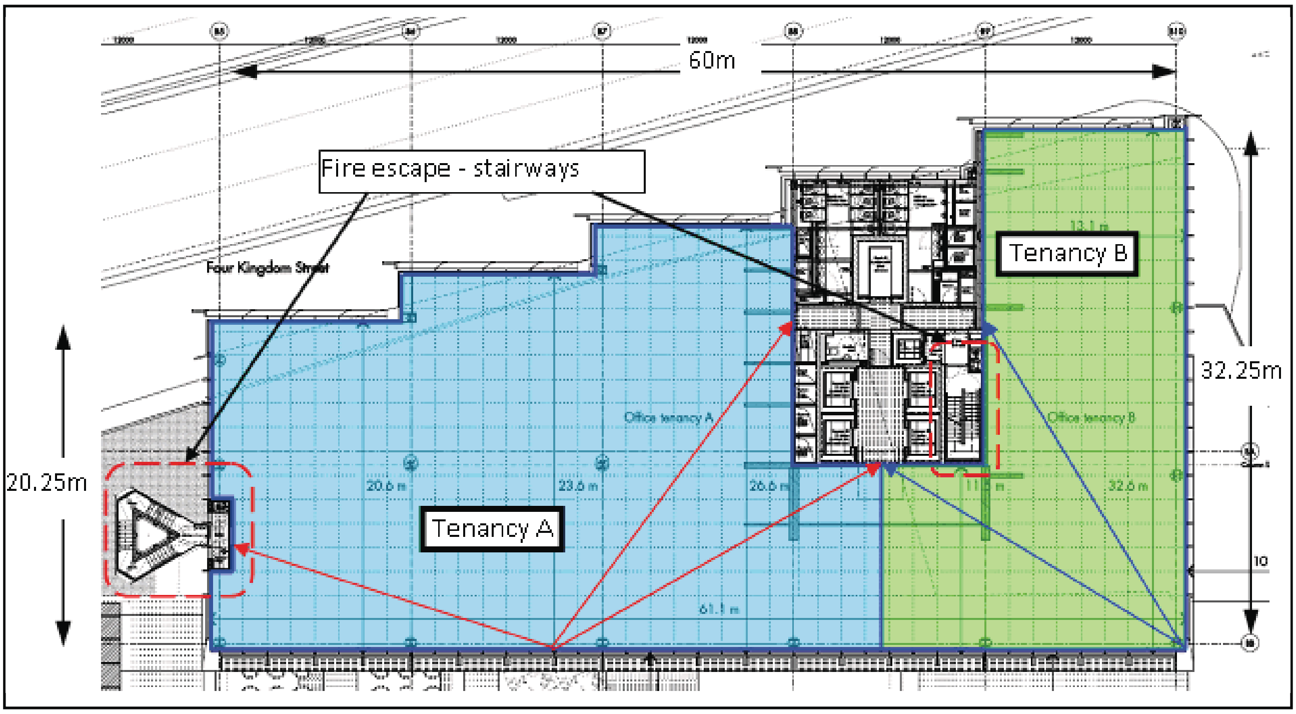


Fig. 6.3 Split-tenancy means of escape

### 6.2.2 Fire escape and fire-fighting strategy

Figure 3 shows the two tenancy areas on a typical office level. In general, the development follows the guidance and recommendations of ADB (2006) in support of the Building Regulations and the London District Surveyors' Association's Section 20 guidance (LDSA 1997). The building is to be sprinklered in accordance with British Standards (BS EN 12845 2004+A2 2009) requirements, and will follow a phased evacuation in the event of a fire. The fire floor will initially be evacuated, with subsequent evacuations two floors at a time. The method of escape in the event of fire is proposed to be through a protected stairway at the centre of the building and an external stair at the west wing of the building (Fig. 6.3). Occupants escaping via the central stair will discharge at basement level before exiting the building via a protected corridor; occupants escaping via the west external stair will discharge at podium (ground) level via an exit which is independent of that from the ground floor office space.

Two fire-fighting shafts are included in the development, with only one protected fire-fighting lift. This is considered acceptable because the main fire-fighting shaft, with the fire-fighting lift, is located close to the middle of the floor plate, so that all parts of the floor can be reached within a 60m radius. The fire-fighting lobbies are ventilated, and all rooms opening into the fire-fighting access corridor are preceded by a protected corridor, with rooms of special fire risk preceded by a lobby with 0.4m$^2$ of permanent ventilation.

Simulation of occupant evacuation was conducted using STEPS, to predict physical movement of occupants into and through the escape routes, based on the split-tenancy internal layout shown in Fig. 6.3. A worst-case total occupant load of 235 is assumed, on the basis of a 1 person/6m$^2$ occupant density for office use (ADB 2006). Due to the low occupant loads of the basement and deck levels, evacuation at these levels was not included in the simulation. Figure 4 illustrates the evacuation of the fire floor.

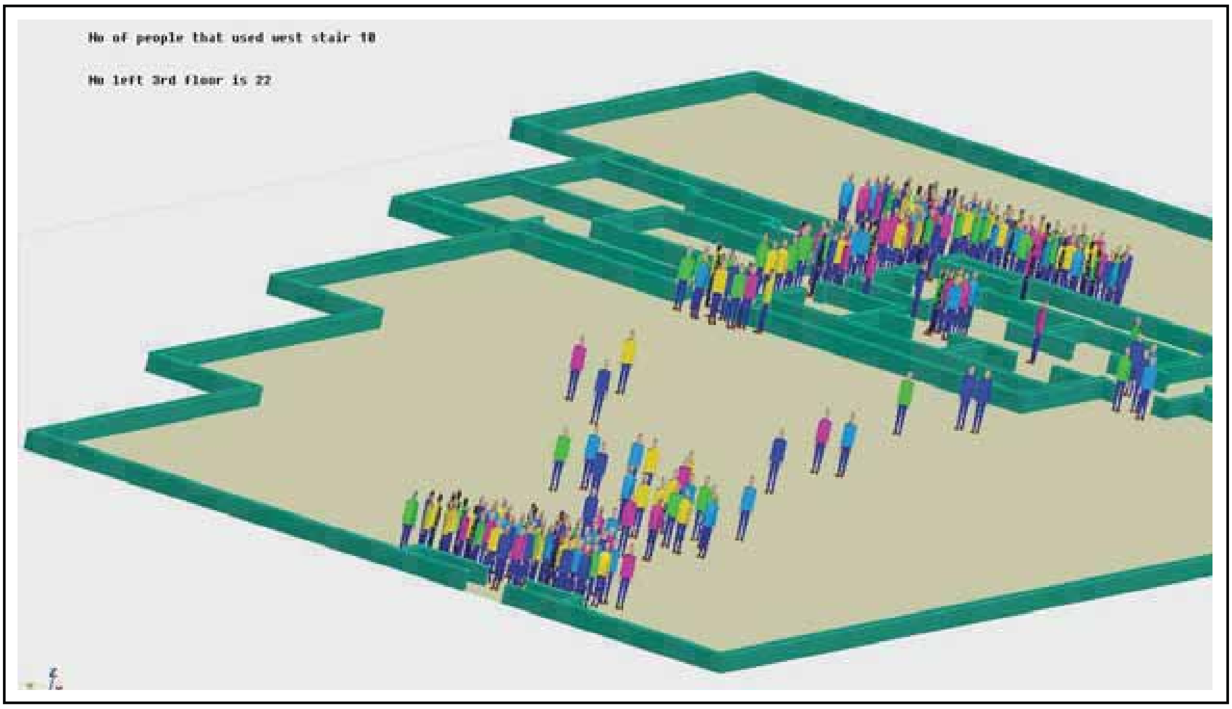


Fig. 6.4 Simulation for occupant evacuation – Typical floor level

### 6.2.3 Structural fire engineering

In the event of fire, the temperatures reached in a compartment and the duration of a fire are directly dependent on the ventilation in the fire compartment. When a fire reaches the stage where there is ignition of all the combustibles within the compartment, the intensity of the heat in the hot smoke layer will cause glazing and non-fire-resisting façades to fail. This allows hot gases to escape through openings.

Traditionally, the fire resistance times specified in most building regulations are based on the Standard time-temperature curve (BS EN 1991-1-2 2002), which does not represent any type of natural building fire, but represents a more severe heating condition than that experienced in many typical natural fire compartments. Moreover, recommendations in ADB (2006) for structural fire resistance do not consider ventilation conditions. Therefore in order to take advantage of the features of the building, the "equivalent time of fire exposure" method is adopted to relate the severity of the natural fires which might occur to the time-temperature relationship in a Standard fire test. In addition, the knowledge provided by recent research in the field of structures in fire has been used to provide alternative solutions for passive fire protection in fire compartments. Global FEA of relatively large subframes allows engineers to examine the structural behaviour of a composite steel frame as it continues to support loading at the Fire Limit State. In many cases this type of analysis can be used to validate a reduction in the number of steel beams that require passive protection, or a reduction in protection, whilst ensuring that structural stability and compartmentation are maintained. This analysis highlights areas where the structure is less robust during a fire, and suggests where additional fire protection or structural measures may need to be introduced.

Fire development

The development of a fire can be predicted by several methods. In this study, a representative compartment was selected for evaluation of the equivalent time of fire exposure. As shown in Fig. 6.5, a tenancy area on a typical office level was modelled as a case study in this paper. The design fire load was determined according to Eurocode 1 Part 1-2 for this representative fire compartment, taking into account the active fire-fighting means provided by sprinklers, to reduce the fire load density. The ventilation factor depends on the area of external façade glazing likely to fracture and provide ventilation to the fire compartment. The thermal properties of the compartment lining depend on the type of floor and wall insulation system. Taking into consideration the degree of conservatism which is necessary for taller buildings, a multiplication risk factor of 2.0 (for buildings more than 30m high) was used. The results indicate that a "time-equivalent" period of less than 60 minutes is safe for this office building. This indicates that the performance-based approach to the design fire itself may show that reduced periods of fire resistance are sufficient in meeting the required level of fire safety, and thus in limiting the temperatures reached in the structural members.

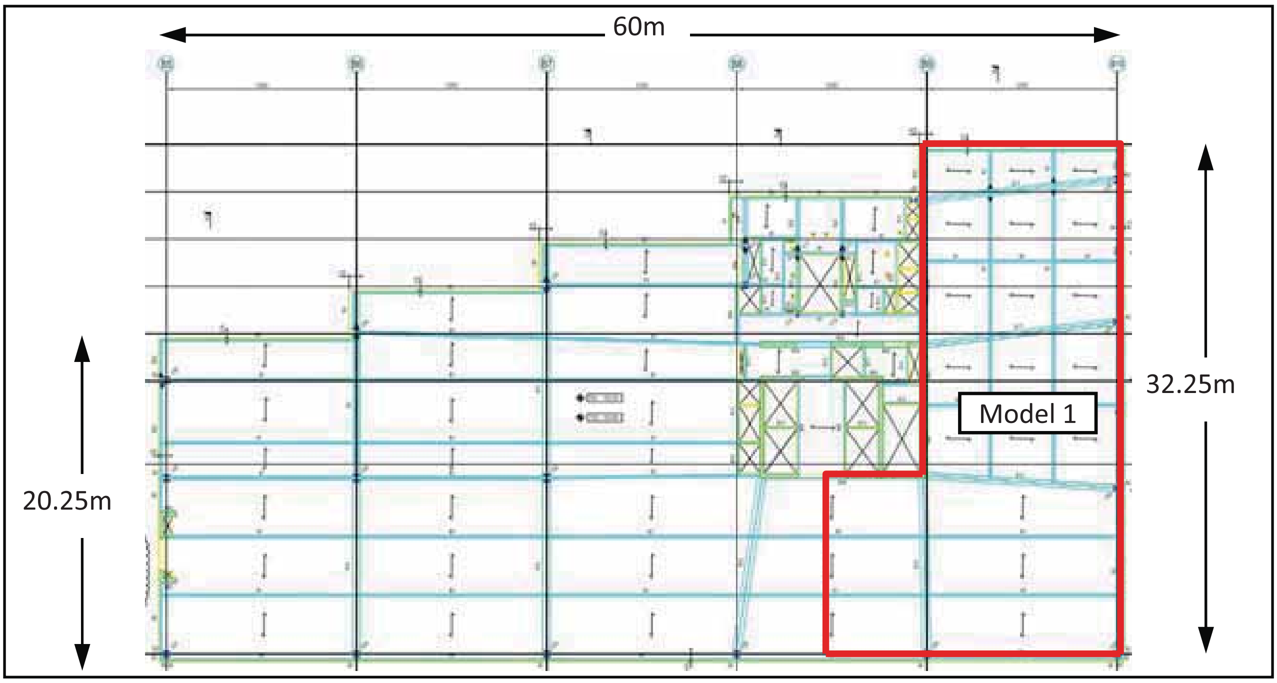


Fig. 6.5 Typical floor plan - Compartmentation Model 1

Structural response – the FE model

The Standard fire (BS EN 1991-1-2 2002) was used with the reduced period of exposure in predicting the effects of fire within a global model, within which most of the secondary beams in the fire compartment were left unprotected. An FE model was developed using *Vulcan* to predict the structural behaviour in fire of a typical floor plate. *Vulcan* does not directly allow the modelling of beams with web openings. As a result, a conservative equivalent section was assigned to the whole length of each beam, with a web area equal to the net web area at the largest opening. This assumption guarantees that the net cross-sectional areas remain constant, and the stiffnesses of the sections are conservative. To evaluate the results provided from the FEA, the following criteria were applied:

- All structure within the fire compartment should maintain its stability, integrity and insulation throughout the entire fire resistance period;
- To allow membrane action to develop in the composite slab, vertical support to panels is achieved by protected beams around the perimeter of each panel, which will in general coincide with the column gridlines. Therefore, protected beams which bound slab panels should maintain their stability at all times during a fire;
- In accordance with the specifications stipulated in BS 476-21 (1987), no rapid increase in the rate of deflection should happen in any region of the floor plate within the prescribed fire resistance period.

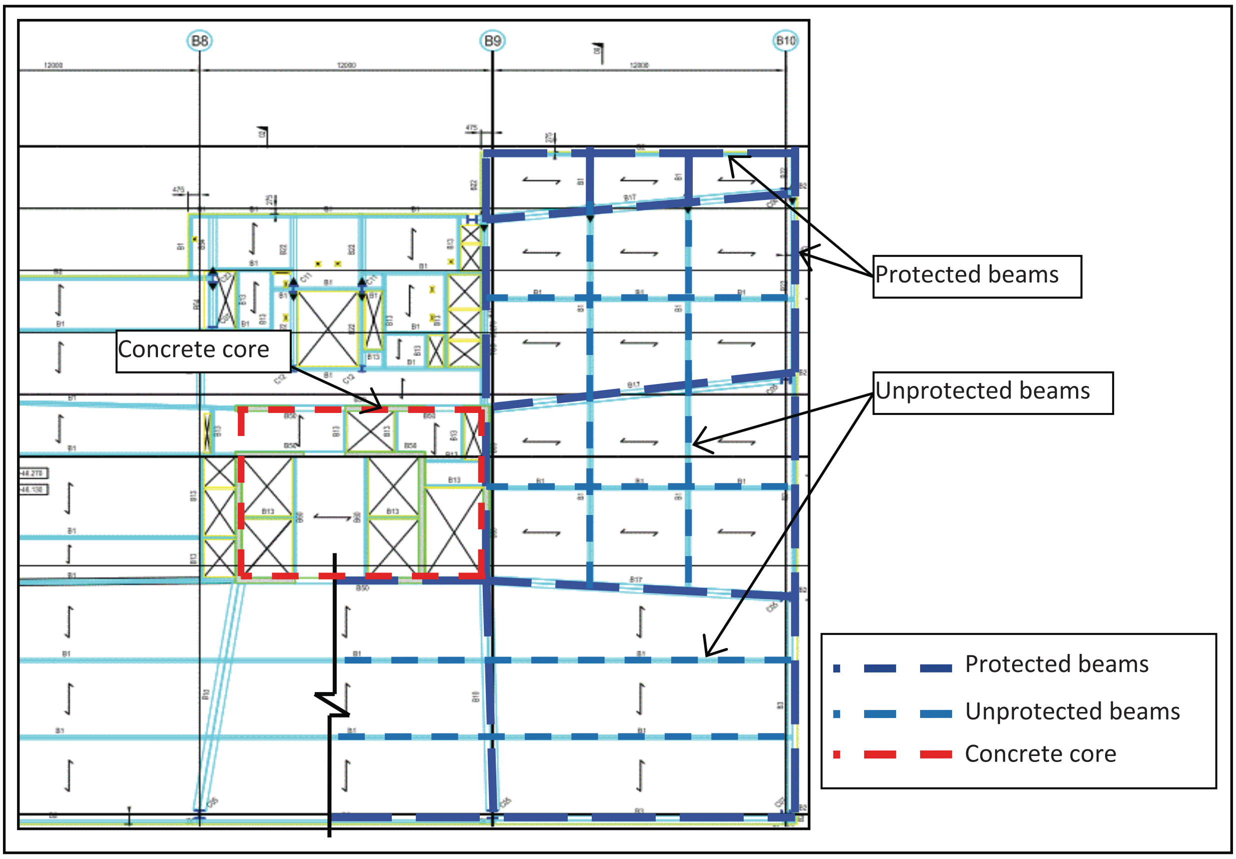


Fig. 6.6 Fire protection scheme of Model 1

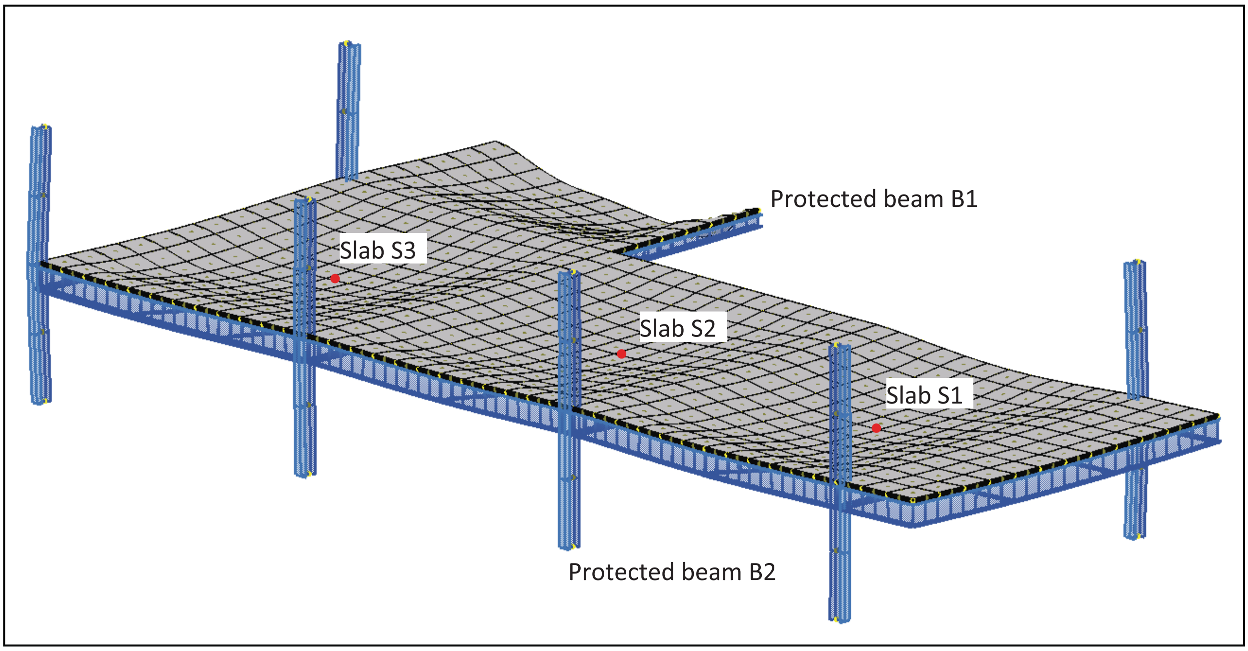


Fig. 6.7 Model 1- predicted global model at elevated temperatures

Fig. 6.6 shows the proposed protection strategy for the fire compartment. Beams are 12m long, with most of the secondary beams left unprotected, while all the main perimeter beams and columns are fully protected. The heating regime in this Standard fire analysis is based on the assumption that the protected steel columns will reach a maximum temperature of about 550°C at the end of the fire resistance period. This is based on the prescriptive UK requirements for fire resistance (ASFP 2010). Beams which contain openings have structural failure modes which are very different from those of normal solid-web beams; therefore, separate fire resistance checks were carried out to provide the limiting temperatures for any section design, taking account of the nature of the critical stresses.

Outcomes of numerical modelling

Fig. 6.7 shows the predicted global fire behaviour of the numerical model. The mid-span deflections of the unprotected concrete slab panels (S1, S2 and S3) and the protected beams (B1 and B2) are presented in Fig. 6.8. None of the slab panels suffer from the loss of strength of the unprotected secondary beams at the end of 60 minutes' fire exposure. No runaway structural failure was observed in the beams or slabs.

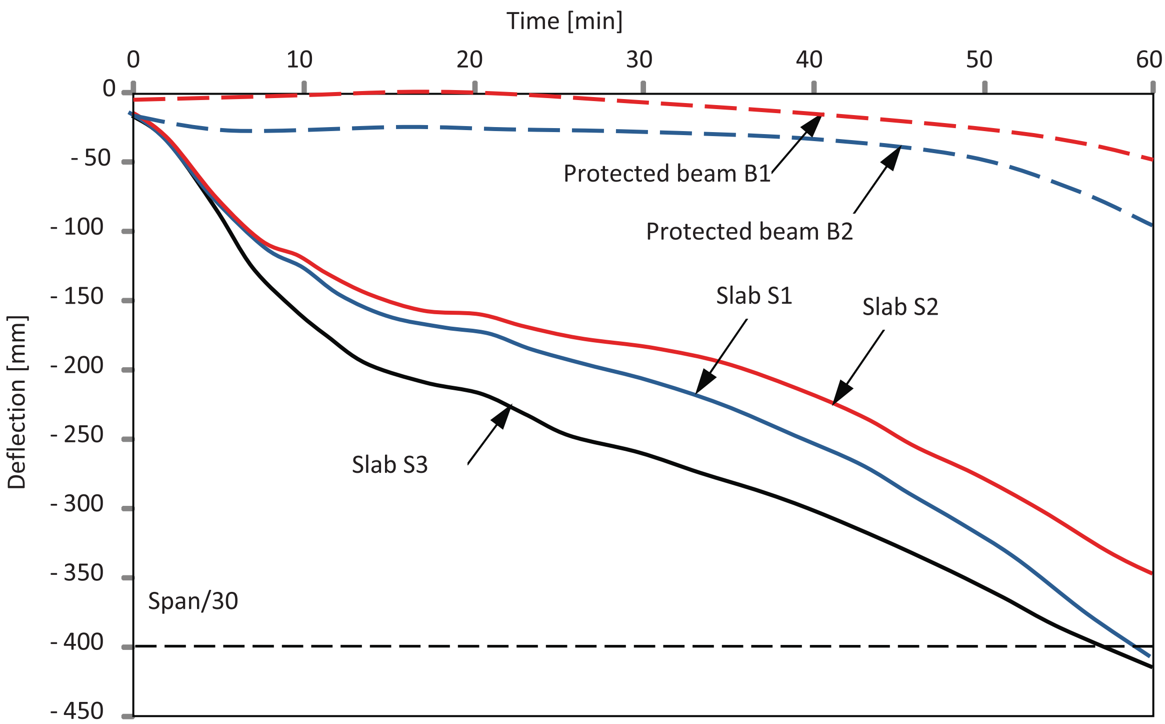


Fig. 6.8 Mid-span deflection

## 6.3 CONCLUSION

A detailed FEA of the structure within a compartment was carried out to predict the global behaviour of the structure under exposure to a Standard fire. It was clearly demonstrated that the performance-based structural fire engineering solution is able to provide an efficient way of increasing the accuracy of modelling of the real structural behaviour in fire. The performance-based approach showed that strategically placed passive fire protection for a composite office building can satisfy the functional requirements of England and Wales Building Regulations, as well as leading to savings on the project cost by optimising the requirement for passive structural steel fire protection.

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# 7 RELIABILITY OF STEEL ROOF STRUCTURES OF THE SPALADIUM SPORTS HALL IN CASE OF FIRE

Summary

In this paper a method for analyzing fire resistance of load bearing structures in big indoor spaces, based on Eurocode, is presented through case study of the steel roof structure of the Spaladium Sports Hall in Split, Republic of Croatia. The temperature-time relationships for indoor space are obtained by the zone model numerical calculation which is applied for two characteristic fire situations: fire in the arena, and fire on the grandstand. The reliability of the roof structure is analysed through the temperature-time curves in load-bearing elements of the steel structure. Parallel to zone model calculation mentioned above, CFD numerical modelling of smoke propagation and determination of temperature fields under the roof structure is performed, which gives additional and more accurate input data for analysing roof structure reliability as well as enable the reduction of costs for passive measures of fire protection.

## 7.1 GENERAL BUILDING DESCRIPTION

Spaladium Centre, sports and business complex (Fig.7.1) is located on the northern part of the Split peninsula and it was prompted by the Men's World Handball Championship in January 2009 when Split was one of the host cities. The centre is designed by “Studio 3LHD” from Zagreb, Croatia and consists of a multifunctional handball arena with 12,000 seating capacity, sports, recreation and wellness centre, a shopping center of 30,000 m$^2$, a parking garage with 1,500 parking spaces, a 100m high office tower, and a sky bar withan exclusive restaurant on the top floor.

Construction of the complex is planned through phases. The first phase has been carried out in 12/2008 and implied a Spaladium Arena while other facilities are planned to be constructed through the second phase.

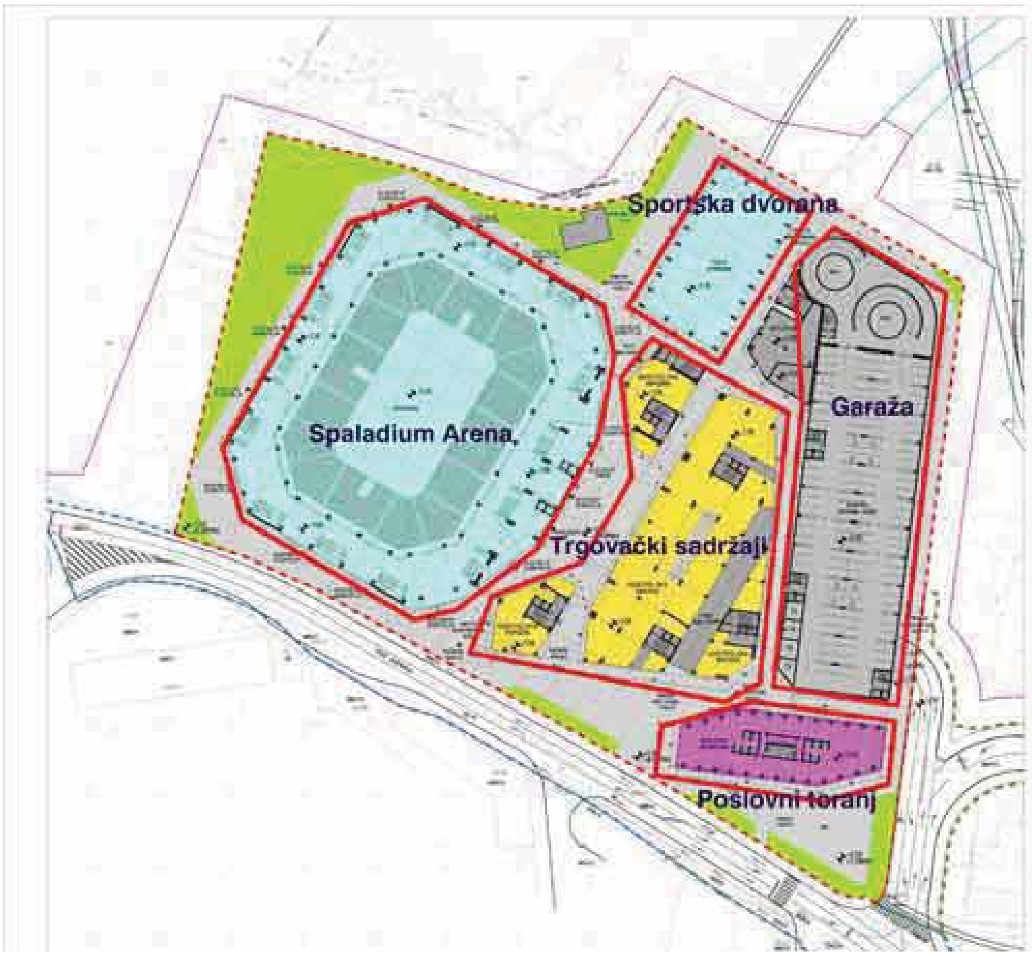



Fig. 7.1 Plan of Spaladium Centre

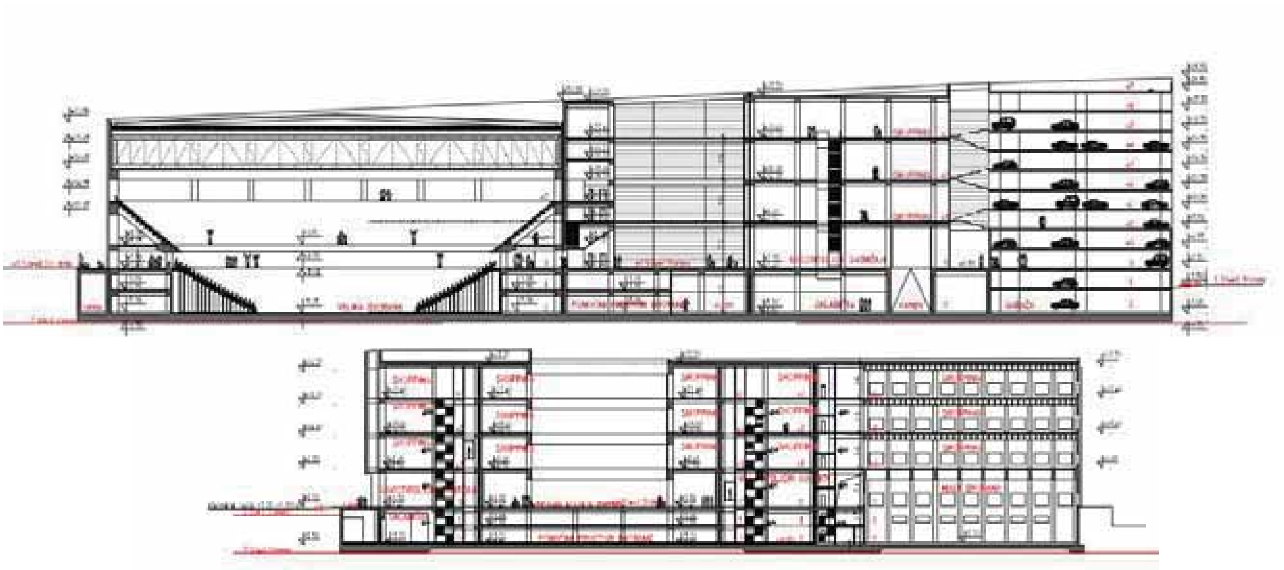


Fig. 7.2 Cross sections of Spaladium sports hall

SPALADIUM ARENA (Fig. 7.2 and Fig. 7.3) is a multi-purpose hall with a gross surface area of 28.500 $m^2$ and a 12,000 seating capacity. It has squared plan with dimensions of 80,3 x 100,3 m and height of 30,25 m. Clear span from the ground floor to the bottom chord of lattice girder is 24, 57 m. On the rooftop of the sports hall there are 30 smoke air vents, used for emergency venting of the smoke in case of fire. Fire protection study (Bezic, 2007) indicated that the estimated fire load would be approximately 300.0 MJ/m2 in the fire compartment located on the grandstand and the fighting arena. Approaches of the viewers, access control and evacuation has been simplified by placing the main processions around the hall, which is also used as a fire access. On the main processions are 14 stairs which viewers access to the upper stands.

Fig. 7.3 Interior of Spaladium sports hall during sport event

**7.2 REGULATORY REQUIREMENTS FOR FIRE SAFETY OF HALL**

For assessing adequate fire safety of object, the combination of prescriptive measures, requirements of fire authorities and performance based approach was applied. As Croatia has no regulatory fire safety requirements for sports hall, it is allowed to use foreign regulatory requirements as “recognized rule of technical practice” (as described in Croatian regulation). That is the reason why NFPA 101 was used. According to NFPA 101 fire resistance rating for building structure was type II (222). Based on investor’s request, fire authority accepted the fire resistance rating for steel roof of 1 hour due to closed distance of professional fire brigade (500 m) and large number of evacuation stairs (14).

According to Croatian regulation at the time when Spaladium sport hall was designed (2008.), the usage of performance based design was not clearly defined yet. From 2010 year on the performance based design could be used in accordance to new Fire protection law. Despite these facts, fire engineering methods was used for the roof structure thermal response of Spaladium arena and for the smoke movement prediction in the evacuation phase (generally for the entire building).

**7.3 GENERAL ASSESSMENT STRATEGY**

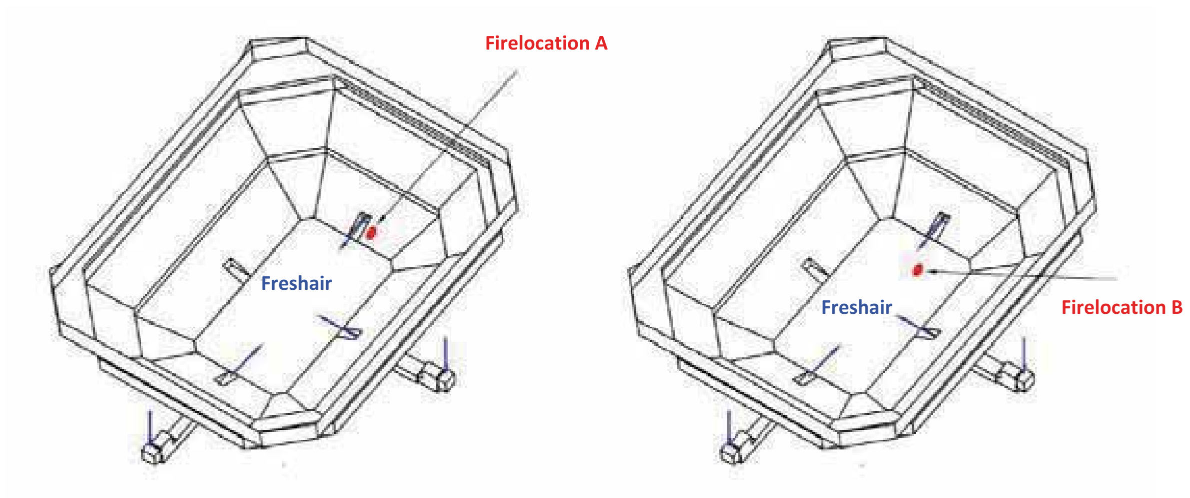
The concept for the truss girders of steel roof structure was as follows. The first step was to determine the fire load for particular sport building (300.0 MJ/m$^2$). Using the t$^2$-method, the fire was simulated in a two locations using zone-model software JET. Finally, the compartment temperatures were used as thermal action in several thermal finite-element-simulations including steel cross sections and intumescent coatings to predict the steel temperatures. The load bearing capacity at t=60 min was calculated using the method of the critical temperature and where necessary using methods of simplified mechanical calculations. The efficiency of natural smoke exhaust system by air vents was proved using the CFD-model FLUENT.

### 7.4 COMPUTATIONAL FLUID DYNAMICS

In order to predict the smoke propagation during the entire period of evacuation, proving that the smoke layer will not endanger the evacuation routes (minimum height of non-smoke zone at the highest point of stand of 1,83 m) the computational fluid dynamics (CFD) was used.

For the Spaladium Arena sport hall fire scenario for two locations was used:

a) grandstands
b) ground floor (playground)




a) b)

Fig. 7.4 Fire scenario locations with main fresh air intakes

In the case b) it is presumed that fire risk is pretty low (almost no combustible materials), but in the case of fire possible smoke production is large because of the height of predefined "non smoke" zone. This fire scenario can be also applied for the central positioned stage. In the case a) it is presumed that fire risk is higher (main combustible materials are seats) and consequences are more serious because of the presences of visitors. In the both cases "axisymetric plume model" is used according NFPA 92B with "volumetric" fire source.

Selected design fire for the smoke propagation calculation by CFD modelling is represented by the “t-square” fire curve, described as:

$$Q(t) = \alpha \cdot t^2 \qquad (1)$$

Choosing the growth coefficient of fire $\alpha$ = 0,045 kW/s$^2$, fast fire scenario is predefined [Hu, 2006]. Time period for firemen intervention (input data from the Fire Protection Study) is defined in 300 seconds, so the resulting heat release rate (HRR) peak is 4,05 MW (Fig. 7.5).

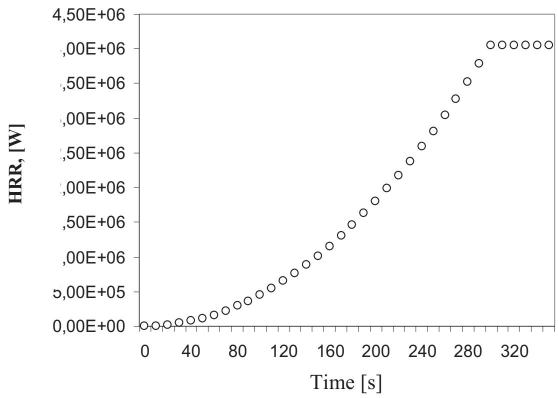



Fig. 7.5 Time-HRR curve for design fire (max. HRR=4,05 MW)

From our point of view, selected "t-squared fast fire" peaking at 4.05 MW is applicable for this project because it is based on full-scale experiment results, but probably more conservative in our case, using the fast fire growth (experimental fire growth is reported "to be fluctuated between slow and medium").

### 7.5 STRUCTURAL FIRE ENGINEERING

For the calculation of the fire resistance of the structure, the two most extreme fire design scenarios were also considered (Fig. 7.6):

- Fire occuring at the centre of ground floor (playground)
- Fire occuring at the edge of the grandstand.

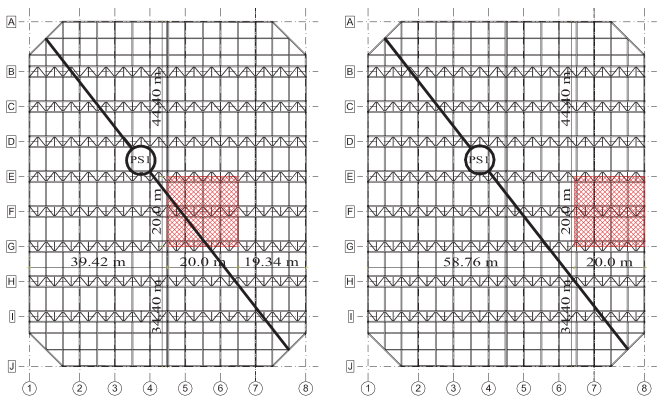



Fig. 7.6 Two considered fire design scenarios

Fire occurring in an enclosure such as a sport hall goes through three distinctive phases: the growth phase, the phase of fully developed fire and the decay phase. HRR curves are used to describe the time dependent release of heat caused by a fire source inside the compartment. These curves were calculated under the assumption that 70% of the original fire load would be consumed in the growth and development phase of fire, while the remaining 30% of the fire load will be consumed in the decay phase of fire. The calculated HRR curve was used as an input parameter for a fire model JET (Davis, 1999) and its graphical presentation is given in Fig. 7.7. JET is a two zones, single compartment model designed to predict the plume centerline temperature, the ceiling jet temperature and the ceiling jet velocity produced by a single fire plume. The impact on the upper layer due to the presence of draft curtains, ceiling vents and thermal losses to the ceiling are included in the model. The unique feature of this model lies in the fact that the characteristics of the ceiling jet depend on the depth of the hot layer.

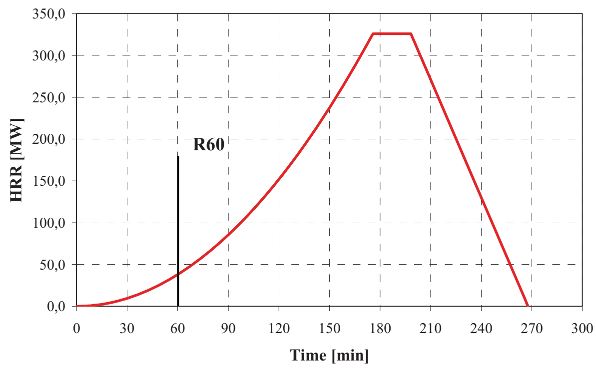



Fig. 7.7 Input HRR curve

Fig. 7.7 shows that the growth phase occurs during the first 170 minutes, which is a characteristic of a slow burning fire. It is also evident that the required fire resistance of roof structure is within that period, which makes the growth phase of the fire the most relevant one.

Fig. 7.8 (a and b) presents the obtained parametric temperature-time curves in the area of the structure ceiling, just below the origin of fire.

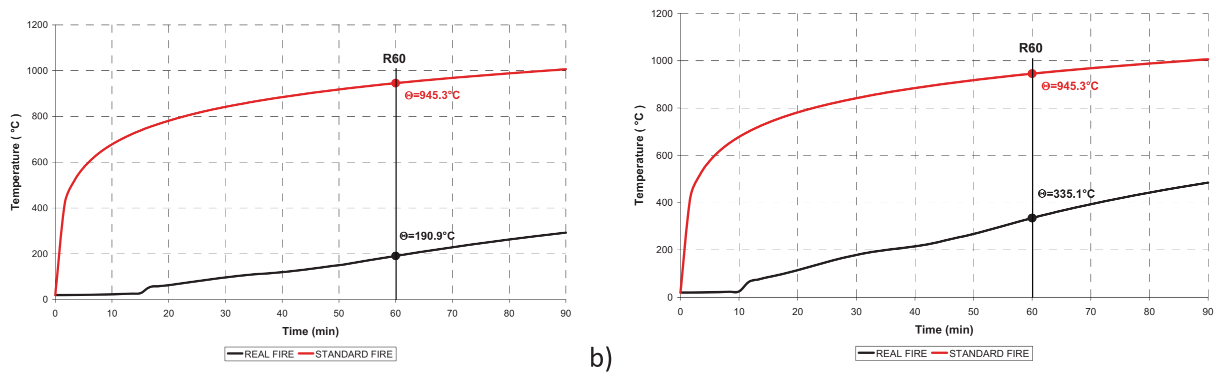
a) 


Fig. 7.8 Parametric temperature-time curve a) H = 11.0 m – upper chord of lattice girder
b) H = 6.5 m – lower chord of lattice girder

### 7.5.1 Description of the heat transfer analysis

In order to analyze the distribution of the temperature inside the structural elements, a heat transfer model was used. The transfer of heat through the structural elements was modeled by a two dimensional transient, non-linear heat transfer model TASEF (Temperature Analysis of Structures Exposed to Fire) (Sterner, Wickstrőm, 1999). Derived temperature-time curves (Fig. 7.9) were used as boundary conditions to solve the heat transfer equation. Results of the heat transfer analysis for the selected steel elements are given in Fig. 7.9 – 7.10.

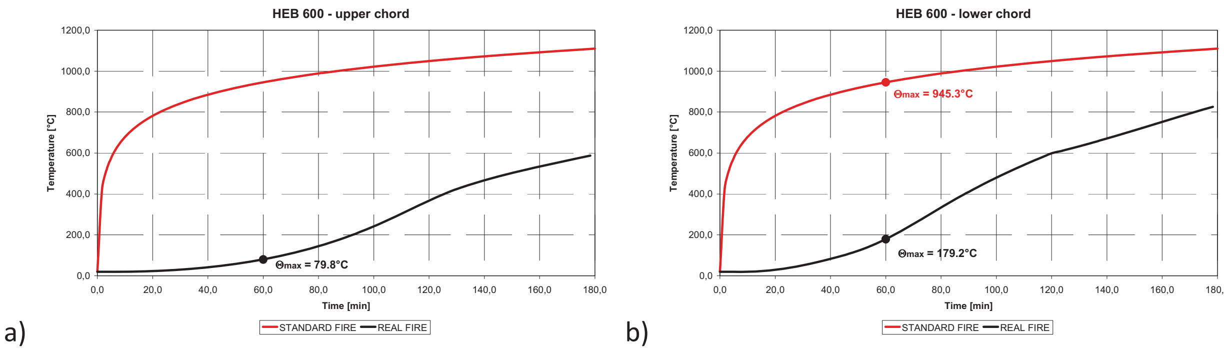



Fig. 7.9 Derived temperature-time curves in cross section: a) in the upper chord of the lattice girder (GP); b) the lower chord of the lattice girder (DP)

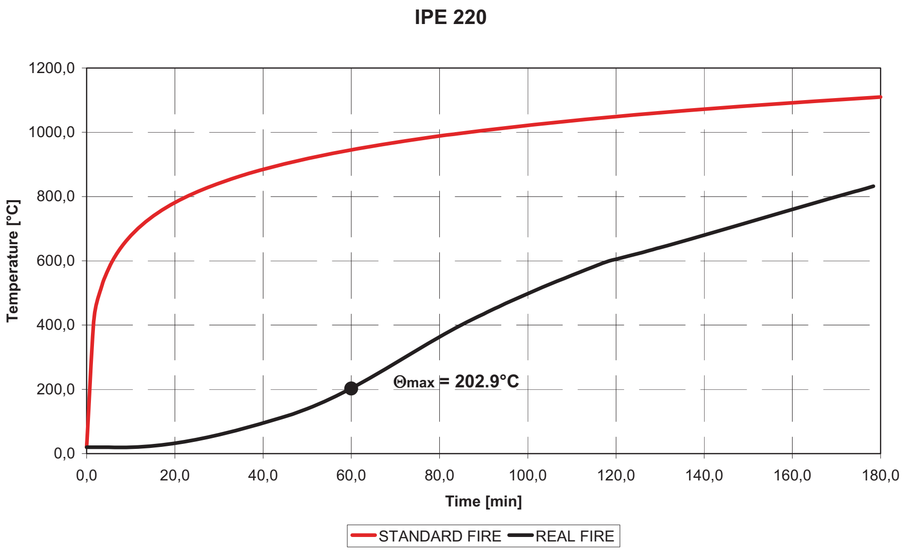



Fig. 7.10 Derived temperature-time curves in cross section in the diagonal element of lower chord of the lattice girder (H2)

### 7.5.2 Thermal response of the structure

Thermal response of the roof structure was analyzed on a segment, while the rest of the structure was replaced by a system of elastic springs with adequate stiffness representing the neighbouring lattice girders (Fig. 7.11).

In addition to the external forces that act as a load on a roof structure, temperatures occuring during fire were added as a load on a part of the lattice girder, depending on the fire scenario, which causes additional internal forces in the elements of the lattice girder.

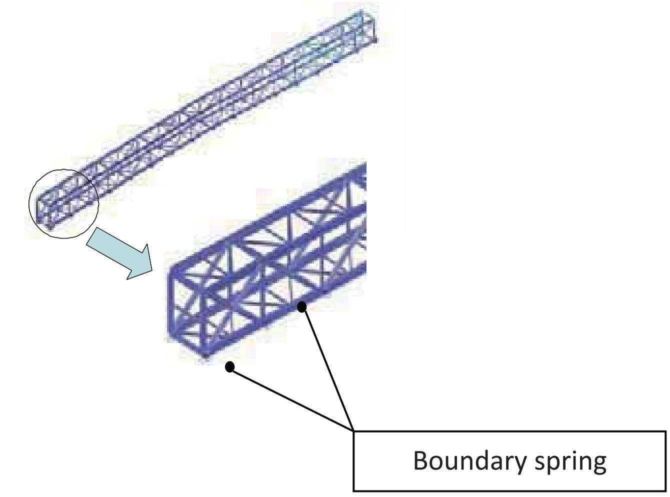



Fig. 7.11 Derived temperature-time curves in cross section in the diagonal element of lower chord of the lattice girder (H2)

**7.5.3 Mechanical response of the structure**

In order to determine the reliability of the structure, two parameters must be defined: structure resistance R (expressed as the ultimate force which causes structural failure) and action on structure S (forces resulting from the structure loads). The steel structure resistance is defined by a series of parameters, the most important being: yield strength of steel, modulus of elasticity, cross-sectional area of the structural element and the moments of the resistance of the cross-sectional area. The action upon the structure can be classified into several load types, the most relevant being: permanent load, variable load, snow load, wind load, earthquake load and fire.

Generally, parameters R and S are characterized by a different number of variables which are mainly non-deterministic. In that case, the probability of structural failure can be expressed as the probability of the event (Milčić, Peroš, 2002):

$$p_f = P\left(G(X) \leq 0\right) = \int \ldots \int_{G(X)\leq 0} f_X(X_1 \ldots X_n) dX_1 dX_2 \ldots dX_n \tag{2}$$

in which *G(X)* is the limit state function which depends on the type of structural failure (failure of cross section or structural element), $f_X$ is the joint probability density function of variables $X_1,\ldots,X_n$. The probability of the structural failure $p_f$ can be determined, depending upon the safety index $\beta$:

$$p_f = \Phi(-\beta) \tag{3}$$

where $\Phi$ is the Standard normal cumulative distribution function.

The integral defined by expression (2) can be solved only by applying complex probabilistic models. The probabilistic model STRUREL was used in this paper for the determination of the safety index $\beta$ for a ''Spaladium'' sports hall for specific cases of the fire load.

Eurocode 1 defines only one limit state wherein the structural failure is supposed to have occurred:

- Ultimate limit state – a state wherein the structure should have sufficient load capacity (resistance) to whitstand the given static load.

The safety proof in case of fire for the ultimate limit state can be defined by the expression:

$$p_{f,fi} \cdot p_{fi} \leq p_t \tag{4}$$

where $p_{fi}$ is the probability of fire action, $p_{f,fi}$ the probability of structural failure resulting from fire action, and $p_t$ the normed probability of the structural failure. Consequently, it can be concluded that the codified value of the safety index $\beta_{fi,norm}$ can be determined from expression (4) for the fire action (ultimate limit state):

$$\beta_{fi,norm} = \Phi^{-1}\left(p_{f,fi}\right) = \Phi^{-1}\left(\frac{p_t}{p_{fi}}\right) \tag{5}$$

Codified probability for structural failure during the period of exploitation, in the case of fire action, depends upon the conditions of evacuation for the structure under study; for normal conditions of the evacuation this amounts to $p_t$=1.3·10-4. Probabilistic variables for resistance R and action S are presented in the Tab. 7.1

Tab. 7.1 Basic resistance variables X

| **Basic resistance variables [X]** | | | | |
|---|---|---|---|---|
| **Variable** | **Mean value** | **Variation** | **Distribution** | **Type of variable** |
| X1 | $k_{y,\theta}$* 41.9 kN/cm2 | 0.054 | Lognormal | Yield strength |
| X2 | $k_{E,\theta}$* 21000 kN/cm2 | 0.0645 | Weibull | Modulus of elasticity |
| X3 | Depends on the cross section | 0.05 | Normal | Cross section area |
| X4 | Depends on the cross section | 0.05 | Normal | Resistance moments Wy and Wz |

where:

$k_{y,\theta}$- reduction coefficient for the yield strength,

$k_{E,\theta}$ – reduction coefficient for the modulus of elasticity.

Tab. 7.2 Basic action variables Y

| **Basic action variables [Y]** | | | | |
|---|---|---|---|---|
| **Variable** | **Mean value** | **Variation** | **Distribution** | **Type of variable** |
| Y1 | $\overline{T}_S, \overline{T}_1, \ldots, \overline{T}_n$ | 0.30 | Gumbel | Fire action expressed as mean temperature in a cross section |

Fig. 7.12 presents the longitudinal and transverse cross section of the segment of the roof lattice girder with the position of the steel elements.

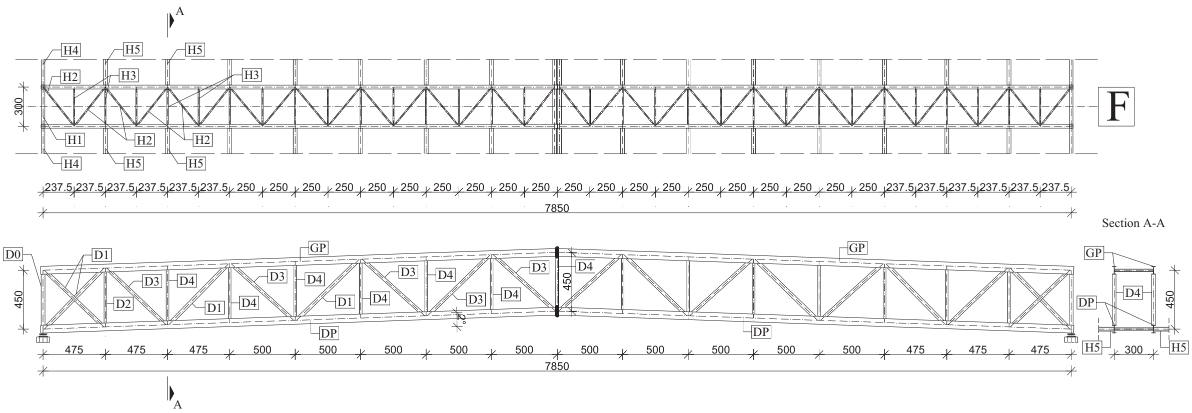



Fig. 7.12 Longitudinal and transverse cross section of the lattice girder section

Tab. 7.3 Position of the basic steel section of the lattice girder

| Position | Profile |
|---|---|
| **GP** | HE 600B |
| **DP** | HE 600B |
| D0 | HE 300B |
| D1 | HE 280B |
| D2 | HE 200A |
| D3 | HE 260A |
| D4 | IPE 200 |
| H1 | HE 300B |
| **H2** | IPE220 |
| H3 | IPE 180 |
| H4 | HE 260A |
| H5 | HE 300A |

Tab. 7.4 presents the results of the calculated safety index $\beta$ and the codified value $\beta_{fi}$ for the characteristic elements of the lattice girder structure.

Tab. 7.4 Position of the basic steel section of the lattice girder

| Position | $\beta$ | $\beta_{fi}$ |
|---|---|---|
| GP | 4.2 | 2.0 |
| DP | 4.0 | 2.0 |
| H2 | 2.2 | 2.0 |

Calculated values of the safety index $\beta$ show that the elements of the load bearing roof structure are higher than the codified values of the safety index $\beta_{fi}$ in case of fire, thus the reliability of the designed structure is proved.

**7.6 CONCLUSIONS**

The main goal of this paper was to describe the challenges which designers were faced with in the situation when national fire regulations are pretty poor and inadequate for particular building. Combination of prescriptive and performance based design was used for the roof structure thermal response and for the smoke movement prediction in the evacuation phase of Spaladium arena, with the aim to optimise the construction costs and to prove important input presumptions regarding the evacuation process. Combination of mentioned methods with extensive usage of numerical modelling didn't reduce the fire safety level; even more it enables cost effective solutions for the roof structure construction and provides safe conditions for the visitor evacuation.

The complete design in the fire safety area was the compromise between the investor, the fire authorities and the entire project team. This compromise request the big efforts from the fire engineering designers during the process of project development, because of the fact that actual Croatian Fire law at that time didn't recognize the performance based methods as the valuable alternative of the prescriptive design approach.

Acknowledgement

The authors would like to thanks the Architectural Studio 3HDL for providing architectural design of Spaladium Arena.

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# 8 REDEVELOPMENT OF ASCOT RACE COURSE - APPROACH TO FIRE SAFETY ENGINEERING

Summary

The redeveloped Ascot Racecourse opened in 2006. The creation of a stadium and an adjacent multilevel space to accommodate larger numbers of people are at the heart of the redevelopment. The varied activities associated with a modern major horse racing facility requires an holistic approach to fire safety including structural performance, smoke management and means of escape. To support this requirement a fire engineering approach was fundamental to deliver the essential safety and viability of densely populated spaces of this nature. The strategy to deliver an adequate standard of safety, background to the risk assessments and supporting analyses are explained to demonstrate what an engineered approach can deliver.

## 8.1 APPROACH TO PERFORMANCE BASED FIRE SAFETY ENGINEERING

There are two primary ways of achieving an adequate standard of fire safety in a building. One is the simple application of building codes and standards, which require limited engineering as the majority of solutions, are prescribed. There is little flexibility. Alternatively, a fire safety engineering approach gives greater design flexibility to achieve a particular performance but requires greater skills involving analysis, risk assessment and engineering judgement. There is often an opportunity to improve value and or performance by selecting the most appropriate combination of fire protection measures with each building requiring its own consideration and its own solutions. Engineered solutions can also be used to demonstrate an equivalent level of fire safety where there is a variation from prescribed guidance. As the complexity and analytical techniques advance the engineer is well positioned to lead this process.

The fire engineering design of the new Ascot Race Course facility is an example where an holistic engineering approach was adopted to deliver the benefits, the value and to eliminate the fragmentation between disciplines that manifests itself on many projects. It is the combination of the built provision, the operational procedures, the fire service response and the identification of realistic fire scenarios that delivers the most appropriate cost effective solution. In the case of Ascot this was effectively co-ordinated by the fire safety team which consists of all the main stake holders.

The basic approach involved making sure that there was sufficient time to evacuate the many occupants of varying age and ability at any stage in the racing cycle throughout the day. This involved assessing that the spread of smoke and the performance of the fire protection measures were sufficient to establish that the means of escape was reasonable. However the built provision alone was not sufficient and fire safety management was equally important.

**8.2 DESCRIPTION OF THE PROJECT**

Ascot Authority (Holdings) Ltd is responsible for running Ascot, the UK's premier racecourse. The largest event is the Royal Meeting which is a five day event, with up to 80,000 spectators of all ages across the social spectrum visiting each day. The previous facilities were becoming dated in terms of the need to meet the demands of a modern busy racecourse. The decision was therefore taken to re-align the track and build a new facility to reduce congestion and create an exceptional facility fit for many years of racing, hospitality and entertainment. The design comprises a series of facilities facing the course, to the front, and parade ring, to the back, with a Galleria running throughout. All the boxes, balconies and facilities make maximum use of the track side of the Galleria, which means that stairs and other facilities are positioned away from the track side, which bring their own fire and circulation challenges.

Fig. 8.1 Grandstand

**8.3 METHODOLOGY FOR MANAGING THE FIRE SAFETY TEAM**

In major complex projects, like Ascot Race Course where the normal national codes have little real relevance a performance based engineering approach is essential. This required the designers, the operators and all the relevant authorities to come together early on in the process to define the technical approach and processes. It was only in this way that viable safe quality spaces could be achieved. Advanced engineering methods were used as typical available design codes were unlikely to deliver the same flexibility or consider the essential levels of safety. The main participants in the fire safety team were: -

- Client
- Fire and rescue services
- Project managers
- Building control
- Structural engineers
- Crown property representation
- Architects
- Sports grounds authority
- Building Services engineers Third party checkers
- Fire engineers

With this number of parties involved, the sequential development of the engineering in parallel with a progressively advancing approval was the key to successful understanding and risk reduction. The fire engineering approach adopted was exemplified by the following steps as part of the overall engineering methodology.

1. Initiation and development of fire
2. Smoke management
3. Structural response and fire spread
4. Detection and alarm
5. Fire service intervention
6. Means of escape

Steps 1, 2, 3 and 6, which are the most significant in engineering terms, are dealt with in this paper.

## 8.4 INITIATION AND DEVELOPMENT OF FIRE

Determination of the size of fire and its speed of development has an impact on all aspects of fire safety. Therefore early agreement was essential as it has an impact on all of the following.

- Structural fire performance requirements
- The need for added fire protection to steelwork
- Spread of smoke and smoke management
- Means of escape
- The requirement for sprinklers
- The extent of compartmentation
- The degree of management to control the fire load content

- Fire service response and provisions

All of these issues were very interactive and thus the agreement of fire size was a fundamental precursor to the engineering, the risk assessment and the expert opinion, which was managed via the fire safety team process.

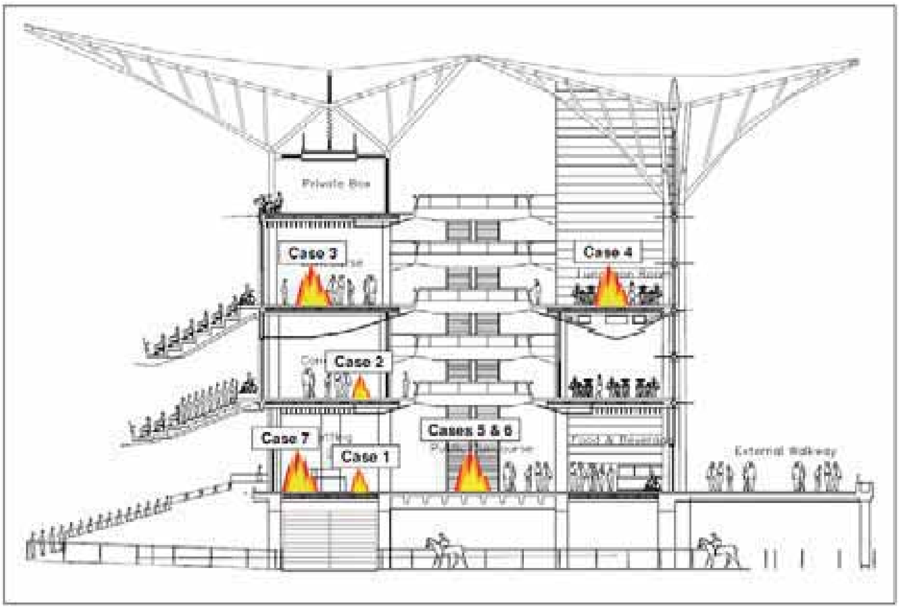


Fig. 8.2 Fire scenarios – indicated on section through Galleria

The range of principal fire scenarios identified in consultation with the fire safety team is illustrated in Fig. 8. 2.

- Sprinklered fires in retail and exhibition areas from 1.25 to 2.5MW with medium to fast growth rate fires. The main influence was on means of escape with little impact on structural performance although integrity (spread of smoke through small cracks) of structural walls and floors remains a relevant performance requirement. (Cases 1 & 2).
- Full developed non-sprinklered fires in high risk areas (e.g. storage) contained by compartmentation so mainly impacts on structure and compartmentation. (Case 7).
- Fires on open decks with no sprinklers, managed fire load and medium to fast growth rate fires from 3MW to 6MW. The primary impact was on means of escape but local temperature checks required on the structure to make sure that a relatively small fire does not have a disproportionate impact on structural performance. (Case 4).
- Separately compartmented tunnel access under the stand with sprinklers, fire resistance and smoke control to reduce risk of fire affecting the business of a major race event.
- Christmas tree. (Case 5)

### 8.5 SMOKE MANAGEMENT

The development of the form of the roof and the integrated design of the smoke vent layout (Fig. 8.3) was very important in the strategy for limiting smoke spread and maximising smoke extraction to allow the longer means of escape time required for the large populations. Account was taken of the wind effects and the microclimate that dominate the flows of cooler low buoyancy smoke. The wind regime around the building, including the impact of dominant low level openings was investigated by wind tunnel testing to establish that there were no significant inflows through the vents, which would prevent venting of the smoke (Fig. 8.6). Also different seasons were considered to test a range of micro climates before the fire scenarios were imposed on the analysis.

For fires on high level floors (Case 3and 4) the smoke calculations show that there is a clear layer above the highest occupied level of about 5m to 7m thus allowing means of escape. The CFD analysis (figure 5) showed that there is sufficient buoyancy to enable this approach.

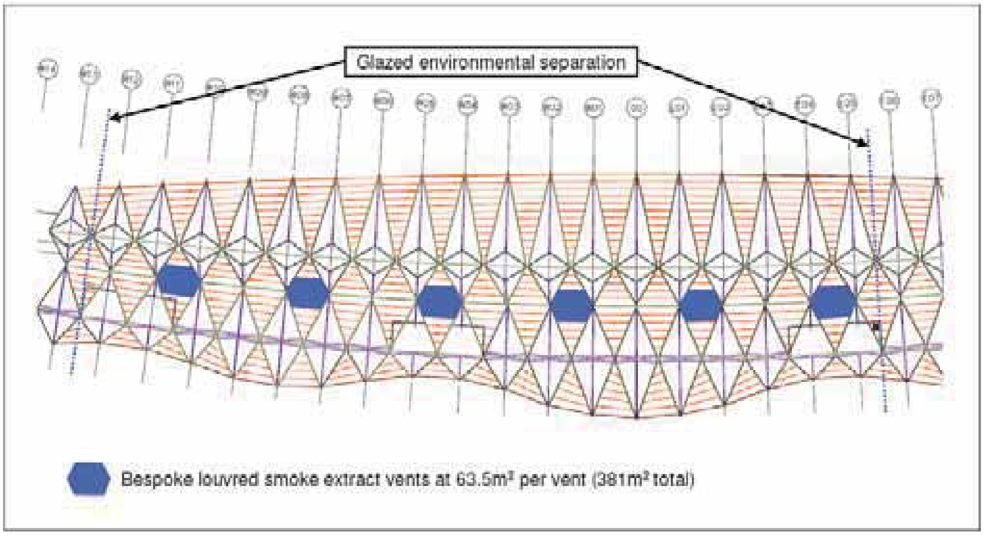


Fig. 8.3 Roof plan - showing layout of vents at natural high points in roof

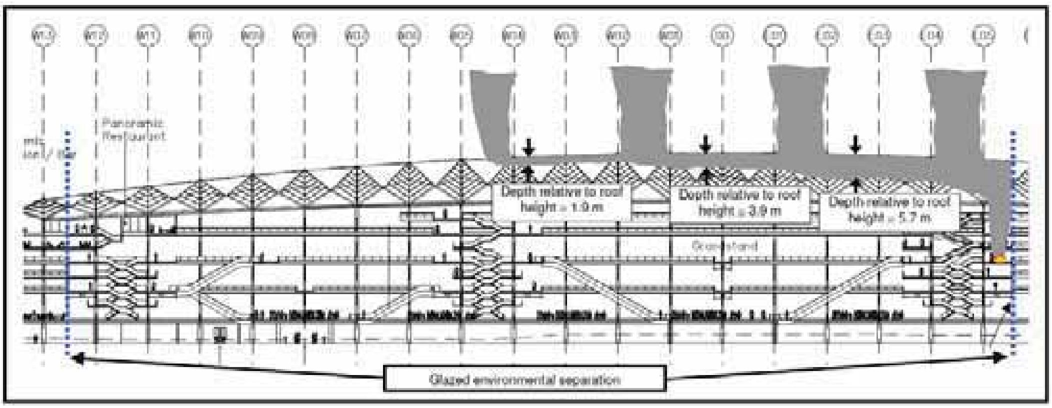


Fig. 8.4 Long section through Galleria - showing plug-holing effect of vents preventing lateral spread of smoke

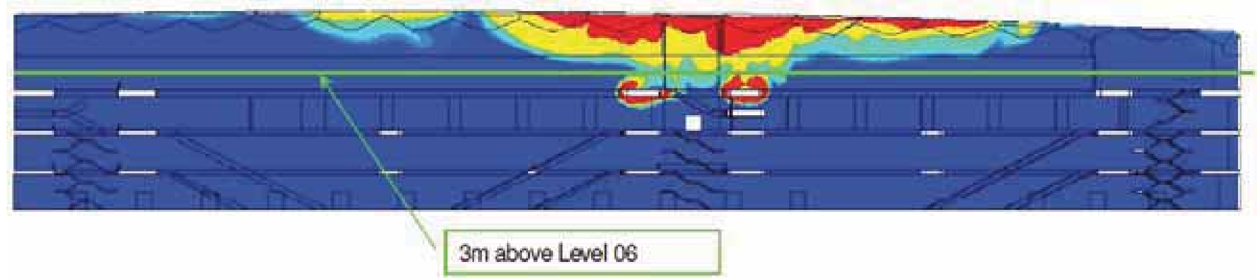


Fig. 8.5 CFD output - demonstrating effectiveness of vent system accounting for microclimate and wind effects

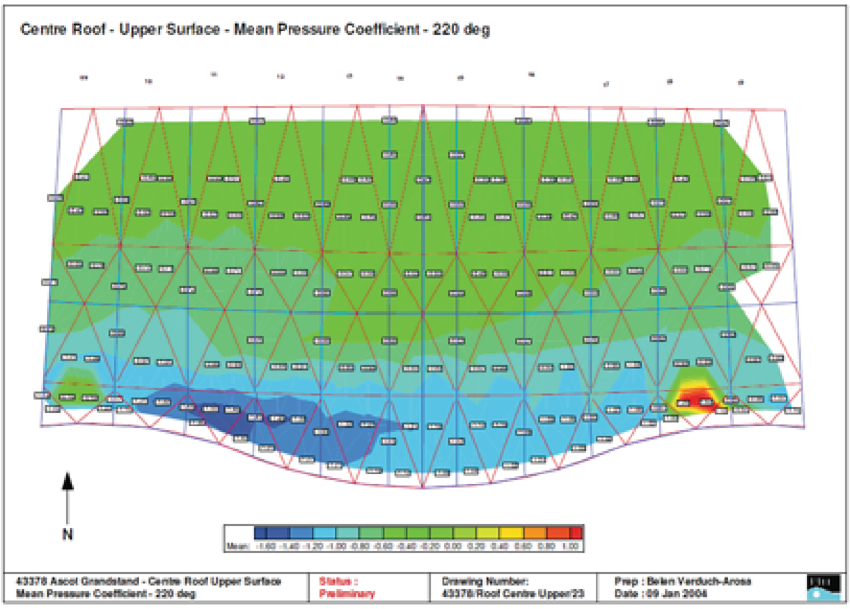


Fig. 8.6 Wind pressure contours – wind tunnel test output supporting location of vents and roof structure profile

For fires on low level floors with sprinklers (Case 1 and 2) the smoke was so dispersed that the visibility was well above the limit normally associated with a dilution system (an approach that allows low density, low risk smoke in occupied areas). The calculations were steady state so there was no time limit built into the calculations, which was conservative. The extent of significant smoke was relatively small with the limits of the smoke plume defined by a 10m visibility iso-surface (see Fig. 8.7). The majority of the Galleria had a visibility well in excess of the 25m, which compared favourably with the normal 10m limit. Christmas trees give very intense fast growing fires but were not a critical case because of the short duration of the fire and the relatively small impact they had on a large space like the Galleria.

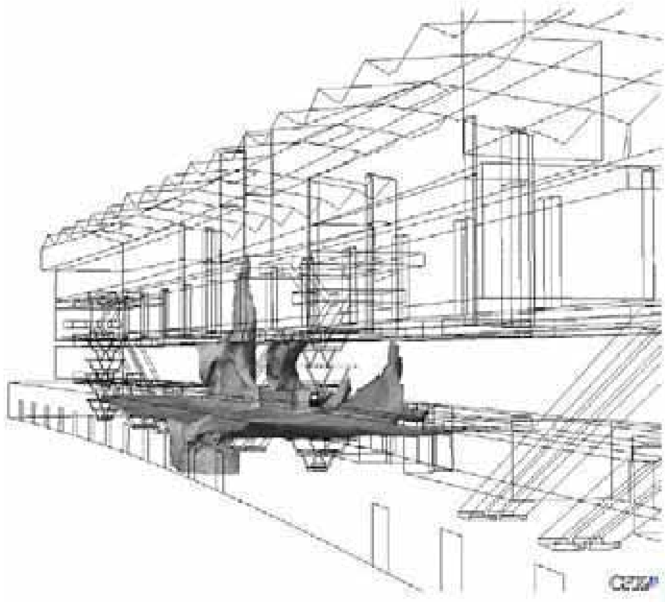

Fig. 8.7 CFD output at 10 minutes – shows 10m visibility iso-surface demonstrating limited extent of thicker smoke and thus which stairs are likely to be affected

## 8.5 STRUCTURAL RESPONSE AND FIRE SPREAD

The elements of structure supporting the proposed Ascot Grandstand were required to achieve 90 minutes fire resistance according to prescriptive guidance. However a fire engineering assessment was adopted to rationalise the applied passive fire protection to the steelwork to test that: -

- Structural elements do not fail prematurely when exposed to fire
- Disproportionate collapse does not occur, and
- A local fire does not adversely affect the structural stability of the overall frame.

A qualitative risk assessment was conducted to determine the appropriate level of applied fire protection considering the following parameters:

- The **probability** or likelihood of an element of structure being exposed to a fire that is sufficiently hot to cause significant structural damage. Consideration was given to the use (fire load density) and size of the space, the openness of the galleria and the effects of automatic sprinkler protection etc.
- The **consequence** of the failure of a particular element of structure to the entire stability of the structure. The consequence of life safety for both building occupants and fire fighters were taken into account. Consideration was also given to property protection / business continuity.


Fig. 8.8 Main structural steel components

The following table was the basis of the fire resistance requirement

Tab. 8.1 Fire resistance matrix

| Fire Resistance | Low Probability | Medium Probability | High Probability |
|---|---|---|---|
| **Low Consequence** | 0 minutes | 0 minutes | 30 minutes |
| **Medium Consequence** | 30 minutes | 60 minutes | 90 minutes |
| **High Consequence** | 60 minutes | 90 minutes | 90 minutes |

Definitions used: -

**Low Consequence** – local distortion of a single element which may lead to higher deflection. However, collapse of any part of the structure is not anticipated and life safety of any occupants and fire fighters are not affected.

**Medium Consequence** – local failure to an element or part of a structure may occur. There may be excessive deflection and/or local collapse at a later stage of the fire. However, major collapse caused by the local failure is not anticipated and life safety of any occupants and fire fighters are not affected.

**High Consequence** – Major collapse or structural instability may occur as a result of the fire.

**Low Probability** – Steelwork exposed to fire sterile area or remote from/external to any significant fire load (e.g. roof steelwork supporting the PTFE)

**Medium Probability** – Steelwork in the proximity of fire load that is protected by sprinklers or other automatic suppression system.

**High Probability** – Steelwork in the proximity of fire load that is not covered by any automatic suppression system.

This approach resulted in the fire resistance ratings given below.

Tab. 8.2 Resulting fire resistance requirements to structural members

| Sample Steelwork Location | *Probability / Consequence* | Typical level of Applied Fire Protection |
|---|---|---|
| **EAST AND WEST END ROOFS** | | |
| DIAGRID ROOFS | LOW / LOW | 0 MINUTES / NOT REQUIRED |
| GABLE TRUSSES | MED / LOW | 0 MINUTES / NOT REQUIRED |
| **NORTH AND SOUTH BALCONIES (Outside Private Boxes)** | | |
| BALCONY BRACKETS | MED / LOW | 0 MINUTES / NOT REQUIRED |
| BEAM CAST IN PLATES | MED / LOW | 0 MINUTES / NOT REQUIRED |
| **EAST AND WEST BALCONIES** | | |
| EXTERNAL BALCONY BEAMS | MED / LOW | 0 MINUTES / NOT REQUIRED |
| INTERNAL BEAMS AND COLS | MED / HIGH | 90 MINUTES - INTUMESCENT PAINT |
| VERTICAL BRACING | MED / HIGH | 90 MINUTES - INTUMESCENT PAINT |
| **TERRACES** | | |
| TERRACE SUPPORTS | LOW / MED | 30 MINUTES - INTUMESCENT PAINT |
| BACKSPAN STRUTS | MED / HIGH | 90 MINUTES - INTUMESCENT PAINT |
| **GALLERIA** | | |
| GALLERIA BRIDGES | MED / HIGH | 90 MINUTES - INTUMESCENT PAINT |
| GALLERIA STAIRS | HIGH / LOW | 30 MINUTES - INTUMESCENT PAINT |

## 8.6 MEANS OF ESCAPE AND RISK ASSESSMENT

For each of the fire cases there was the potential for a stair or a walkway to be blocked and so a series of evacuation studies were carried out accounting for these potential blockages, which were estimated from the extent of the smoke plumes (see Fig. 8.7). The analysis demonstrates there was no real upper limit on the evacuation time so means of escape was not critical. Therefore an in depth assessment of time to detection, pre-movement time (the time taken for people to start moving) was not required. Therefore a figure of 8 minutes taken from the Guide to Safety at Sports Grounds was used to ultimately determine the acceptability, even though there is currently some debate on the relevance of this particular figure. Means of escape for disabled people was planned to be via a combination of refuges at the stairs and also more effectively via horizontal evacuation to the outside areas at the ends of the stand.

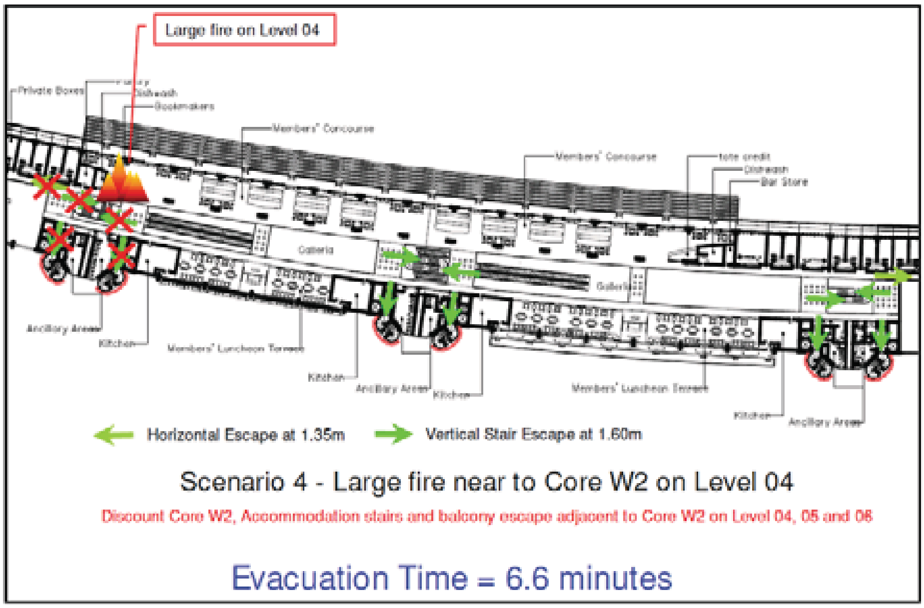


Fig. 8.9 Evacuation scenario - showing blocked stairs and primary exits available for escape

## 8.7 CONCLUSION

This paper summarises the fire engineering assessments undertaken for the redevelopment of the Ascot Racecourse. The creation of a stadium and an adjacent multilevel space to accommodate larger numbers of people are at the heart of the redevelopment. The varied activities associated with a modern major horse racing facility requires an holistic approach to fire safety including structural performance, smoke management and means of escape. To support this requirement a fire engineering approach was fundamental to deliver the essential safety and viability of densely populated spaces of this nature. The strategy to deliver an adequate standard of safety, background to the risk assessments and supporting analyses are explained to demonstrate what an engineered approach can deliver.

Acknowledgment

The authors would like to thank the Ascot Racecourse Ltd for their permission to publish this case study.

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# 9 FAULTY DESIGN OF A SPORT HALL

Summary

Considered building covers a sport hall with social and recreational facilities, and a hotel part. The primary function of the building is to organize mass sport events. The building was designed in 2009 as foreseen for about 2,300 users (including the auditorium for 2,000 people). This two-storey building rises 15.3 m above ground level and its floor area is 6116 m2. Due to the usage and the number of persons who may reside in the facility, it is equipped with a variety of fire protection solutions within both the structural design and technology. The building was designed in breach of the requirements posed by technical regulations, governing the construction and fire safety in Poland. At the project stage there were committed several errors which have a significant impact on the safe evacuation. The design errors were duplicated during the construction phase and led to difficulties with gaining the official acceptance for usage.

## 9.1 GENERAL DESCRIPTION OF THE BUILDING

The subject of the study is a localized in a small town (51 000 inhabitants) a sport - cultural center consisting of the sports hall and the hotel part. The facility is intended to be used for mass events, sporting or cultural (eg, concerts). In the Hall's ground there is a court with the audience for two thousand people and hygienic sanitation facilities (changing rooms and showers) as well as auxiliary storage rooms. In addition, functionally related parts of the sports hall located on the entresol, provide recreational functions implemented in the rooms designed for gym, exercise, fitness, wellness centre, and a restaurant. The facility also includes a two-storey hotel, office space, and a conference room, see Fig. 9.1.

There is also a two-story hotel part in the building. It contains 26 beds and offices and a conference room, which may be occupied by about 27 people.

The building's structure is designed as consisting of a concrete part cast in situ and a steel part. The roof structure is made of prefabricated glued timber elements. The facade walls consist of glass curtains on a steel structure, and masonry walls. Building height is 15.3 m above ground level. The area is 6116 $m^2$ and building area about 4300 $m^2$. The plan of the object is an irregular shape with dimensions of approximately 78m x 65m.

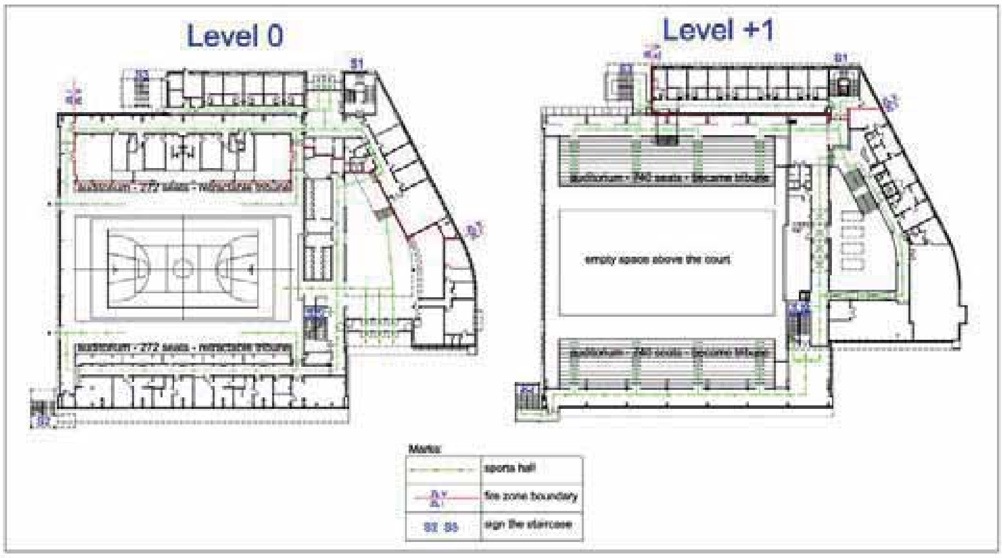


Fig. 9.1 Plan of the building

## 9.2 FIRE SAFETY ENGINEERING SOLUTIONS USED IN THE FACILITY

In terms of passive protection the considered facility was designed as reinforced concrete structure with required fire resistance applied and steel structure protected by fire protection paint. The object was divided into two fire zones (Tab. 9.1 and Fig. 9.1). The need for two zones was due the surface limitations for fire zones, specified in the regulations (Dz.U. 2002 vol.75) (5000 m$^2$).

Tab. 9.1. Fire zone distribution

| Zone No | Name | Description |
|---|---|---|
| Zone I | Sport hall | The main entrance to the building, sports hall with the audience, changing rooms and toilets facilities are under one of the stands some of the rooms, and service loft. |
| Zone II | Hotel part | Hotel rooms on the ground floor and first floor of the building, communication and ancillary facilities within the zone as well as changing rooms and storage areas functionally connected to the sport located below the one from the stands. |

It is difficult to justify why the locker room and sports magazines located under one of the stands were functionally associated with the hotel part (zone II). This division according to the authors was not the optimal solution, and also forced the need for additional passive solutions (additional and unwarranted costs). According to the authors a better solution would be to treat the building with different functions (hotels and sport) as a separate fire zones.

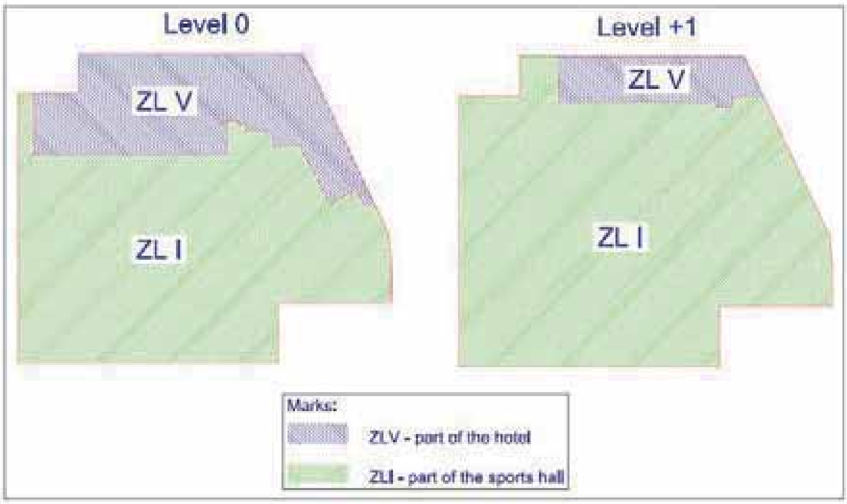


Fig. 9.2. Fire zone distribution

The facility is equipped with the following active fire protection:

- fire alarm system (initially the total facility protection was assumed while in the implementation phase some storage facilities were omitted),
- audible warning system covering the whole object,
- 10 fire hydrants (6 on the ground floor and 4 on the floor), located in recessed cabinets,
- equipment to remove smoke, only installed in the stairwell at the hotel, smoke extraction is carried out by the smoke vents mounted in the ceiling, no air supply holes are made on the ground floor,
- fire electricity breaker covering the entire building.

From the viewpoint of evacuation the facility is functionally divided into three parts (see Fig. 9.1):

- sports hall - part of the audience, on the ground floor main entrance (east) and the hall with cloakroom and staircase to the floor level (entrance to the stands), on the floor of the stands on both sides of the court, located inside the main lobby bar and recreational area (gym, fitness and wellness room), along with ancillary rooms and a mezzanine on which the entrance is located on the eastern grandstand,
- sports hall - part for the players on the ground floor of the room stands the two changing rooms, showers, storage ancillary sports hall and associated ancillary facilities,
- part of the hotel, in the western part of the building, separated from the sports hall includes space on the ground floor entrance (west) with a reception desk, hotel rooms and part of the club with a conference hall and hotel rooms located on the floor.

In the initial phase of the project the evacuation of was organized as follows:

Sports hall – the part for the audience (see Fig. 9.1):

- stairs in the hall (leading to the eastern exit),
- enclosed staircase and smoke extraction in the parts of the hotel leading to the western exit (K1),

- staircases K4 and K5 leading of the floors in the eastern part of the main hall and on to the eastern exit,
- communication from the mezzanine stairs being "extension" of the cage K4,
- if additional grandstands are placed on the floor, the evacuation of a sports hall provides an additional two outputs of the hall immediately outside the northern half, in the southern part of the main door to the hall and outside on the east and exit doors to the reception hall and hotel further out west exit.

Sports hall - part of the players (see Fig. 9.1):

- the exits leading to the sports hall and further out the through the door in the northern part,
- the exit leading to the main hall and outside through the east exit (the eastern portion under the stands),
- the exit leading to the reception hall and hotel further out through the west exit (the western portion under the stands),
- for the people in the main hall there are two outputs directly to the outside in the northern and southern parts of the door to the main hall and additionally a door leading into the reception hall and hotel further out through the west exit.

Hotel part:

- the enclosed staircase K1 with smoke extraction leads from the floor to floor and then through the reception hall outside the west exit (from the floor in the north),
- the exit directly to the reception hall and further out west exit (from the ground floor in the southern part),
- the exit directly outside the cage K3 (from the ground floor in the north).

The space of the sports hall was taken as a room where there is no evacuation enters. This solution was supposed to ensure the evacuation consistent with effective requirements, but as it turned out, after verification, it required substantial changes.

### 9.3 FORMAL AND LEGAL REQUIREMENTS

In practice the requirements for a building are placed in advance based on classifying its usage and based on its height. Because of the way buildings are used they are divided into (Dz.U. 2002 vol.75): risks to humans, and production - livestock handling. In terms of human risk there are distinguished five categories known as bad, ZLII, ZLIII, ZLIV and ZLV. The considered building is a subject of two these categories:

- in the entertainment - sports part - bad (the building containing a room for more than 50 people who are not regular users of the building),
- in the hotel part - ZLV (collective residence building).

In terms of the building's height there are distinguished five groups of buildings (Dz.U. 2002 vol.75): low (no higher than 12 m), medium-high (more than 12 m to 25 m) high (more than 25 m to 55 m) and very high. The building under consideration belongs to a group of medium-high buildings.

This assignment is the starting point to determine the fire resistance class of the building denoted by the letters A, B, C, D, E, based on Tab. 9.2

Tab. 9.2 Fire resistance of buildings

| Building height | **Hazard category for people (ZL)** | | | | |
|---|---|---|---|---|---|
| | **ZL I** | ZL II | ZL III | ZL IV | **ZL V** |
| *1* | *2* | *3* | *4* | *5* | *6* |
| low (N) | „B" | „B" | „C" | „D" | „C" |
| **medium-high (SW)** | **„B"** | „B" | „B" | „C" | **„B"** |
| high (W) | „B" | „B" | „B" | „B" | „B" |
| very high (WW) | „A" | „A" | „A" | „B" | „A" |

Tab. 9.3 Requirements for the major elements of structure based on the fire resistance class

| **Fire resistance class of the building** | **Fire protection classes for elements of the building** | | | | | |
|---|---|---|---|---|---|---|
| | main supporting structure | roof structure | ceiling slab | external wall | internal wall | roof decking |
| "A" | R 240 | R 30 | REI 120 | EI 120 (o-i) | EI 60 | R E 30 |
| "B" | R 120 | R 30 | REI 60 | EI 60 (o-i) | EI 30[4)] | R E 30 |
| "C" | R 60 | R 15 | REI 60 | EI 30 (o-i) | EI 15[4)] | R E 15 |
| "D" | R 30 | (-) | REI 30 | EI 30 (o-i) | (-) | (-) |
| "E" | (-) | (-) | (-) | (-) | (-) | (-) |

In accordance with above requirements, the bad and the medium-high buildings ZLV shall meet the requirements for Class B of fire resistance. This gives a basis for determining the detailed requirements for fire resistance of individual elements of the building, according to Tab. 9.3. It should also be added that here that the above requirements are not strictly imposed. By applying appropriate solutions for the fire protection some of the requirements set can be released. Additionally, if the building element is also a part of the fire zone borderline, it must meet the requirements for fire resistance specified in Tab. 9.4.

Tab. 9.4 The requirements for building elements forming the fire protection separations

| **Fire resistance class of the building** | **Fire resistance class** | | | | |
|---|---|---|---|---|---|
| | fire separating elements | | fire doors and other closures of fire | fire door from the fire protection vestibule | |
| | walls and ceilings, with the exception of ceilings in ZL | ceiling in ZL | | the corridor and into the room | the staircase |
| „A" | REI 240 | REI 120 | EI 120 | EI 60 | E 60 |
| „B" i „C" | REI 120 | REI 60 | EI 60 | EI 30 | E 30 |
| „D" i „E" | REI 60 | REI 30 | EI 30 | EI 15 | E 15 |

Another requirement, which determines the subsequent functional layout of the building is acceptable surface fire zone. It is determined based on Tab. 9.5.

Tab. 9.5 Permissible surface of fire zones

| Hazard category for people | Permissible surface of fire zone [m$^2$] | | | |
|---|---|---|---|---|
| | In a building with one floor above ground (no height restrictions) | in multi-story building | | |
| | | low (N) | **medium-high (SW)** | high and very-high (W) i (WW) |
| *1* | *2* | *3* | *4* | *5* |
| **ZL I**, ZL III, **ZL IV**, ZL V | 10 000 | 8 000 | **5 000** | 2 500 |
| ZL II | 8 000 | 5 000 | 3 500 | 2 000 |

It is worth noting that the rules (Dz.U. 2002 vol.75) allow the treatment of one building as consisting of many independent if they are properly separated. All elements of separating (from foundation to roof covering the roof, walls, and in particular the closing holes) must then satisfy the imposed requirements (Tab. 9.4).

From the viewpoint of evacuation important parameter is the number of persons who may reside in the building. The parameters of the escape routes are described below. The rooms designed to accommodate people should be prepared for evacuation. This should be achieved by allowing the exit to the outside of the building or to an adjacent fire zone - directly or through overall channels of communication. According to the regulations (Dz.U. 2002 vol.75), there are two parameters of the length of an escape route: crossing routes and access routes.

Additionally, in terms of fire protection equipment in the facility there are required (Dz.U. 2010 vol.109) the following devices:

- fire alarm system,
- audible warning system

- fire hydrants,
- equipment to remove smoke in the stairwells for evacuation,
- electricity fire breaker.

Building under consideration also needs to ensure water supply for fire (Dz.U. 2010 vol.124) - in the amount of 20 $dm^3$ /s and must be equipped with fire roads (Dz.U. 2010 vol.124), providing convenient vehicle access for the fire fighting and rescue units. Furthermore, based on the standards (PN-92/N-01256.01, PN-92/N-01256.02) the building should be equipped with building evacuation and safety signs.

### 9.4 ACCEPTED WAY OF ASSESSING

In this case, the assessment of the solutions covering primarily the verification of safety-related parameters was focused on the evaluation of:

- the length of evacuation pass,
- the length of evacuation access,
- width of escape routes (corridors, stairs and landings),
- total and a minimum width of exit from the premises,
- total and a minimum width of exit outside the building.

Based on the above it was determined how an object is split into fire zones and the distribution of fire stairwells, which can be used for evacuation.

### 9.5 DESCRIPTION OF THE PROJECT APPROVAL PROCESS

The project is being developed by a team of designers of various sectors based on established principles and utility regulations and standards (Dz.U. 2010 vol.243). Then, fire safety in most of the projects is subject to verification by an expert of fire protection. The necessity of this verification shall be governed by (Dz.U. 2010 vol.121) and it is dependent on the usage of the object, its height and area.

An investor verifies the project in terms of functional requirements previously established. The designer is responsible for the project and that it complies with the requirements of the law. Later in the investment process the designer is required to fulfill the author's supervision (Dz.U. 2010 vol.243).

At the stage of building construction the construction manager is responsible for its proper implementation. On completion, he shall make a declaration of conformity of the draft rules of technical knowledge and the law (Dz.U. 2010 vol.243). It is understood by the principle that the contractor should catch any errors in the design and in consultation with the designer, make the necessary changes. With good preparation of construction managers in the design and architectural issues there are observed simultaneously inadequate consideration for fire issues. Sometimes this results in a duplication of errors during the implementation of the design.

The last stage, before the building is open to use, there are various departments involved in the control aimed to ascertain compliance with the construction project (Dz.U. 2010 vol.243). At the same time there is carried out an assessment of compliance with the law. All too often this leads to a detection of serious deficiencies in fire safety at this stage only, and this was the case here.

Before the end of construction, in preparation for the reception of the building for fire protection, the investor asked the independent expert to prepare a document required by the rules "fire safety instruction." Such a document is required in all facilities with a capacity exceeding 1000 $m^3$. This document contains requirements for the fire protection. In the analysis of the object in question, the most important issue raised was concerning exceeded length of the pass and reach exits.

In extreme cases, the length of the transition evacuation (evacuation of the stands), was more than 80 m at the limit of 40 m. In addition, there is an isolated room on the mezzanine floor recreation and wellness center, which meant that these areas should guide the access routes, which was permissible length of 10 m, and in one case 40 m. A similar problem was noted on the evacuation of the floors of the hotel, where the length of evacuation was reaching over 27 m. As these distances are exceeded, it allows to describe the building as a life-threatening for humans. In addition, separation of the room on the mezzanine causes it to be considered as a regular floor.

Another problem concerned here are the staircases. They were not made in accordance with the requirements for smoke (K4 and K5), or even the stairs were open without casing and closing fire door and smoke.

Also the requirement for housing fire escape route and closing holes on the section between the staircase and leaving the building was not satisfied. Moreover, it was noted that according to the requirements, the door of the premises on evacuation routes in the hotel (except the door to hygienic sanitation) should have a fire resistance of not less than T30. This resulted in the need to replace the doors to rooms and spaces under the grandstand and offices located within the same fire zone.

### 9.6 ADOPTION OF SOLUTIONS RESULTING FROM THE THEORETICAL ANALYSIS

According to the project the facility is equipped with additional external stairs (one for each of the stands), located in the northern part of the building (staircase K2 and K3) for the evacuation of a sports hall and the floor of the hotel. This solution eliminated the problem of escape from the stands and the floor in the hotel part. In connection with the requirement of closing the exit from the premises for general communication path in the medium-high ZLV (hotel) building the fire doors needed to be exchanged under the grandstand in the fire zone areas (total 18 units) by the doors with a fire resistance EI 30$^{th}$. It should be noted that it would be possible to avoid part of this exchange if the hotel would be separated from other parts of the building by fire separation walls, which would be treated as a separate hotel building with a height of 9.8 m (low building in which there is no such requirement). The division that has not been foreseen in the draft and at a stage when this requirement was identified as necessary to complete the building was at the finish stage.

It should also be noted that the system used does not guarantee the required area of fire zones in accordance with the requirements, which is less than 5000 $m^2$. However, adequate separation from the sports hall of the hotel building would treat sports as part of the one-story building which would allow the fire zone area to be 10000 $m^2$. Such fire zoning would be compliant with the regulations.

Another problem is related to the wellness facilities and dedicated sports hall on the mezzanine, which in accordance to (Dz.U. 2010 vol.243), is considered as the next story. This problem was solved by dismantling the walls that emit up to the room ceiling, along with separate spaces previously formed as one whole. This work was carried out on a stage adaptation of premises by the user.

The major problem was improper construction of the fire zoning. On the floor at the exit of the sports hall on the escape stairs at the hotel (frame K3) there were installed doors without fire protection. At this point, the designer clearly marked fire zone boundary, but also did not provide fire doors. This error has been discovered after finishing work. There was therefore necessary to replace the exit door of the hotel by fire doors.

As previously mentioned the escape route leading from the closed fire door and the staircase with smoke removal outside the building should have a cover and closing openings meeting the requirements for fire resistance - the same as the staircase (Dz.U. 2002 vol.75). This requirement was not included in any of the staircases provided as the escape way (frames K1, K4 and K5). In the case of the staircase K1 there was possible to make an appropriate enclosure. This required, however, the exchange of 10 pieces of ordinary fire doors. The exchange was carried out at the stage of completion.

In the case of the staircases K4 and K5 the above requirements were not satisfied because the exit of those staircases on the ground floor leads to the space combined with a sports hall. At the same time in these staircases there are no channels connecting staircases with smoke flaps. Finally smoke removal was abandoned for these staircases. These staircases were treated as regular mezzanine staircases connecting the ground floor in the same room. However, the investor has incurred costs associated with the installation of useless control devices, smoke control and smoke flaps.

One year after the completion of the fire protection systems, the inspection revealed irregularities in their operation and performance. Below are some examples that have occurred in the present building:

- no connection control signal from the fire detection system to the system startup smoke dampers,
- no revision allowing access to the detectors located above the suspended ceiling (about 10% of all detectors,
- lack of proper control of ventilation and air conditioning,
- lack of protection (not installed detectors) in several storage rooms,

- some of the evacuation light lamps were not constantly powered to enable charging of the batteries (power supply is connected with the lighting circuit).

Removing these errors and the introduction of the aforementioned changes, will allow bringing the object into line with the requirements of the fire.

**9.7 CONCLUSION**

Any solution for fire safety is costly due to the need to ensure an adequate level of reliability of equipment and due to the process of certification. Moreover, a better effect is to use an expensive material for the interior use of fire equipment, which usually are in such case less visible or they may "spoil" the aesthetics of the object. This means that security is an area that is not popular among investors. It is often not accompanied by an appropriate level of security awareness.

As shown by the considered case, an improper design of the object leads to additional expenditures for fire safety. In this case, the introduced solutions beyond those incurred before putting the building to use will not have a significant impact on the cost of continued operation of the building. No less, it appears advisable to ensure that the proper handling of the planning and implementation of fire safety solutions alongside with the professional architectural design and construction.

Moreover, it is questionable to let conduct reception operations by the contractor. This can lead to concealment of serious shortcomings as described above, lack of access to the detectors mounted above the suspended ceilings, etc..

The significant impact on the correct operation of the investment process has the knowledge and experience of designers and contractors. Remains an open question how an investor should proceed to choose a team to guarantee a comfortable course of investment.

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PN-92/N-01256.02: PN-92/N-01256.02 Znaki bezpieczeństwa. Ewakuacja. [Safety signs. Evacuation].

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# 10 FIRE ENGINEERING CASE STUDIES IN FINLAND

Summary

Fire engineering solutions in building industry in Finland vary depending on e.g. the type of building and of resources used in design. The fire protection can be done by using prescriptive rules or by using performance based design. In this paper, the fire solutions of two quite large commercial buildings in Finland are gone through using the guidance paper done within COST-IFER project as a guideline.

The first building introduced is furniture and household store IKEA built in Finland 2010. The fire solution is based mainly on fire protection with automatic water extinguishers. The design is mainly based on prescriptive rules, but during the design and acceptance procedure the solution had to be validated using performance based design, i.e. additional simulations. The design and acceptance procedure of this will be gone through in this paper.

The other building introduced here is a recreational centre, which is locating in the middle of Helsinki, Finland. The building is ready for use in April 2010. The area of the building is 22200 m$^2$ and the volume is 167700 m$^3$. The fire engineering was carried out using both performance based and prescriptive design. A variety of fire scenarios was simulated using FDS software and the structural analysis was done using the temperatures from these analyses. The simulations were carried out by VTT and the structural analysis by Tampere university of technology together with the main structural designer Finnmap Consulting ltd. The design procedure and the acceptance procedure will be introduced in this paper.

## 10.1 IKEA STORE, TAMPERE, FINLAND, 2010

### 10.1.1 Description

Finland's largest IKEA store was built in 2009-2010 to the district of Tampere. It is the first steel-framed IKEA in Finland. The project made use of Ruukki's structural fire design to ensure fire safety of the steel structures in the event of fire and to choose the most effective fire protection method for the different parts of the construction project. When completed, the largest Ikea store in Finland will have a floor area of 35,000 square metres.

Ruukki Construction is focusing on solutions and systems deliveries that include design and installation. The way of working and technical solutions are cost-effective and speed up the construction process. The building speed plays a big role in the process. Ruukki has earlier been involved in Ikea construction projects in Finland, Sweden, Russia, and Poland.

A consortium of Lemcon Ltd and Rakennustoimisto Palmberg Oy has ordered the frame from Ruukki and Rovakate, which is part of the Icopal Group, has ordered the panels.

| | |
|---|---|
| Storeys: | 1-2 |
| Floor area: | 35 000 m2 |
| Steel structures | -Composite steel-concrete columns |
| | -Tubular steel trusses |
| | -WQ-welded profiles + hollow core slabs in 2-storey parts |
| Wall cladding: | -Sandwich panels (rock wool insulation) |
| Roof structure: | -Corrugated steel sheeting beased solution |
| Fire requirement: | -R60 for load bearing structures |
| Fire protection methods: | -Prescriptive and performance based together |
| | -Automatic water extinquishers |
| | -Intumescent coating |

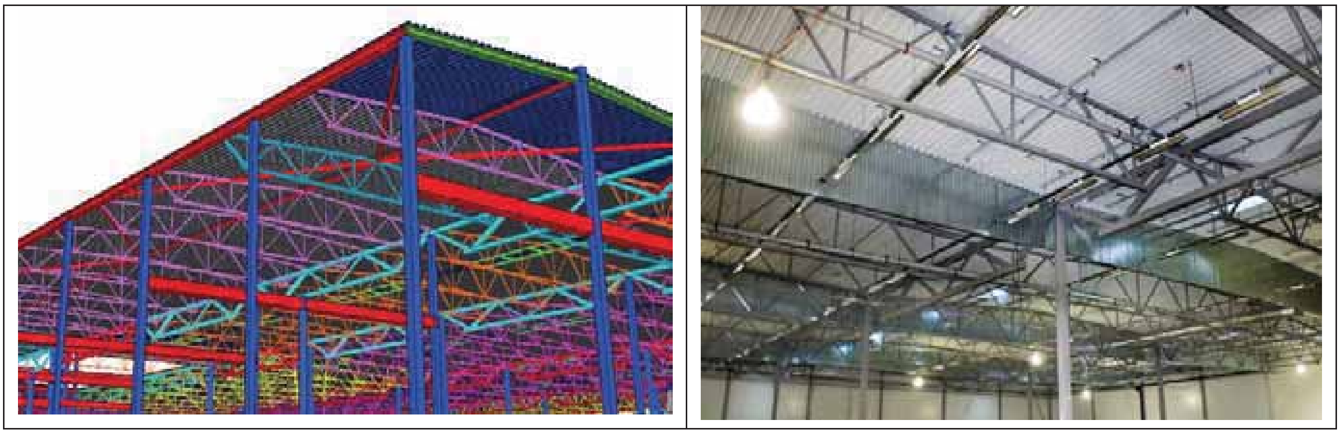


Fig. 10.1 Part of the frame system taken from Tekla Structures model and during the construction

#### 10.1.2 Fire solutions

The building regulations require 60 minutes Fire protection for this case. The Ruukki's national acceptance for "Fire protection of steel structures with automatic water distinguishers" was used as the main fire protection method. With this method one can achieve 90 minute fire protection, having the water flow minimum of 10mm/min. No additional fire protection to steel structures is needed within this system's coverage. The system is optimal for 1-2 storey commercial, large buildings. The approval is based on actual large-scale fire tests and also a large simulation project. The simulation project is still continuing and is carried out by VTT.

Fig. 10.2 Sprinkler protection network on the left. Building site on the right

Part of the structures, especially the bottom chords of the WQ-beams (integrated, welded slimfloor beams) in the intermediate floors were protected by intumescent paint. Also some part of the bracing structures was protected by fire protection materials. This was done in the office area of the building where the spaces are lower, and the water flow from the sprinklers was smaller.

#### 10.1.3 Design and approvals

The fire protection was approved by the building and fire authorities before the building process. Many negotiations were carried out in going into the details of the solution. In Finland the building authorities give the approvals but in this kind of large buildings they always ask statement from fire brigade. So the final solution is done in negotiation with designers and authorities. Also a statement from Research Institute (VTT) was required.

The final installations of sprinklers and also fire protection materials are inspected afterward and documentation is put to the building manual for future inspections and maintenance.

Altogether the fire protection in this case was optimized to different areas using different materials and methods. The performance based design using simulation was used mainly by the 3rd party inspector to ensure that in certain spaces the calculated temperatures of the structures stays at safe level.

### 10.2 SALMISAARI SPORTS CENTER, HELSINKI, FINLAND, 2010

#### 10.2.1 Description

Salmisaari Sports Centre is located in the middle of Helsinki, Finland. The building will be ready for use in April 2010. The floor area of the building is 22,200 m$^2$ and its volume 167,700 m$^3$. The main contractor is YIT Rakennus Oy. The architects and consulting structural and fire engineers are Arkkitehtitoimisto Pekka Lukkaroinen Oy and Finnmap Consulting Oy and L2 Paloturvallisuus Oy, respectively. The load bearing structures were delivered by Ruukki Construction Oy.

The length, width and height of the building are about 136, 35 and 36 metres. There are four stories, each 8-10 metres high. Each storey has a space about 30 m wide supported by 30 m span trusses located at every 5 metres. These trusses are innovative structures used in some Finnish projects: the top chord is made of a welded slim floor box beam that supports pre-stressed hollow core concrete slabs, the braces are of tubular steel and the bottom chord is a flat steel bar. The trusses are about 3 m high. That leaves a lot of space for installations below the floors. The columns supporting the trusses are reinforced concrete filled steel tubes. A general view and the space examined in this study are shown in Fig. 10.3.

Fig. 10.3 General view and the examined space

Performance based fire design was applied in this project only to trusses. Fire actions were determined for the parts of the building topped by trusses. The intended uses of the spaces below the trusses are:

- First floor: two ice hockey rinks (total area 4200 m$^2$).
- Second floor: Bowling, martial arts, restaurants (2000 m$^2$).
- Third floor: Adventure place for children (2000 m$^2$), beach volley (780 m$^2$), badminton (570 m$^2$).
- Fourth floor: dancing (900 m$^2$).
- Climbing wall, area 170 m$^2$, max. height 30 m.

Fire actions were determined for the intended uses of the spaces, and for the following special cases:

- Ice resurfacing machine fire,
- Storage fire with flashover,
- Coat-rack fire,
- Plastic slide fire,
- Stage fire (abnormal use),

Stand fire (abnormal use),

- Climbing equipment fire.

The fire safety plan was prepared by the fire engineers of the project. Fire compartments were partitioned using EI60 structures. The fire compartments consist of stairwells, exit areas, storage spaces, offices, saunas, dressing rooms and special facilities. According to the safety plan, the building should have the following fire safety equipment:

- Initial extinguishing equipment, consisting of: one portable extinguisher per 300 m$^2$ or hose reels.

- Automatic alarm system covering the whole building.
- Smoke extraction, mainly by the fire brigade.
- Automatic sprinkler system.

According to CEA (1998) requirements, the sprinkler system should be able to detect and put out a fire in its early stage, or to restrict the spread of fire until the fire brigade arrives.

Fire actions are determined based on fires which may occur in different spaces (during intended use, special use and abnormal use). The effects of the sprinkler system are taken into account when defining design fires. Traditionally the effects of the actions of the fire brigade and other fire fighting measures are not taken into account in defining design fires. Fire brigade actions are taken into account in the following references: Tillander et al. (2009), Karhula & Hietaniemi (2008), NFPA (1996), Barry (2002) and Hietaniemi (2008).

A summary of the definitions of design fires used in the performance based fire design of this project is given below. More details are given in a report by Hietaniemi (2009). In Finland it is not possible to use Annex E.1 of EN 1991-1-2 (not applicable) to define the fire activation risk due to the size of the compartment and the type of occupancy, which is why probability analysis was used in this study. Fire load densities were determined based on national fire load classifications of occupancies and by conducting a fire load survey using both analysis and synthesis of experimental data as well as modelling and fire simulation.

The fire scenarios and all details of the fire load calculations were approved by the local authorities, the client and the fire safety and structural engineers of the project before fire simulations and structural calculations were done.

### 10.2.2 Fire actions based on intended uses of spaces

The following properties are supposed to be valid for the sprinklers:

- RTI = 110 $m^{1/2}s^{1/2}$
- Activation temperature is 67 °C
- Protection area $A_r$ of one sprinkler is 12 $m^2$.

The defect frequency of sprinklers is 3 % according to International Fire Engineering Guidelines (2005). Assuming a floor area of 5000 $m^2$, and a 12 $m^2$ protection area, about 500 sprinklers are required on that floor. Then the probability is that one of those sprinkler heads is defective. Let us then suppose that this defective sprinkler head is just above the starting point of the fire. The resulting fire scenario would be a so-called local fire in the sprinklered building where:

- The other sprinklers restrict the fire to the protection area of one sprinkler.
- Fire intensity is defined by the use of the space under consideration, as shown later.

Let us then consider the failure of the entire sprinkler system. That can be estimated by the defect flow of Fig. 10.4. The sources of the initial data are the following:

– Pump defect, Isaksson et al. (1998),
– Duct defect and water source defect, Isaksson et al. (1998),
– Installation defect, Korpela (2002).

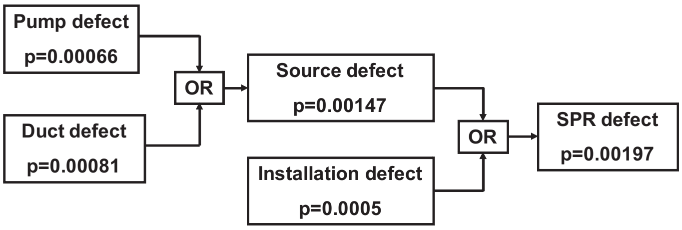



Fig. 10.4 Defect flow of sprinklers

The probability for failure of the entire sprinkler system is about 0.00197 ≈ 0.2 % according to the estimate. On that basis a second fire scenario is created involving a so-called global fire in the sprinklered building:

– After sprinkler activation the fire intensity is doubled from the value defined based on the use of the space at sprinkler activation time and it remains constant. The doubling provides the extra safety required by authorities in this case.

So we end up with two fire scenarios, the first one based on local sprinkler defects and the second one on the failure of the entire sprinkler system. They are graphically demonstrated in Fig. 10.5. In the first case the fire decays either due to a lack of oxygen or combustible material in the space. The fire is local within an area of 12 m$^2$ and should be applied to the most severe locations in the building. The second fire is not dying down and engulfs the whole floor under consideration.

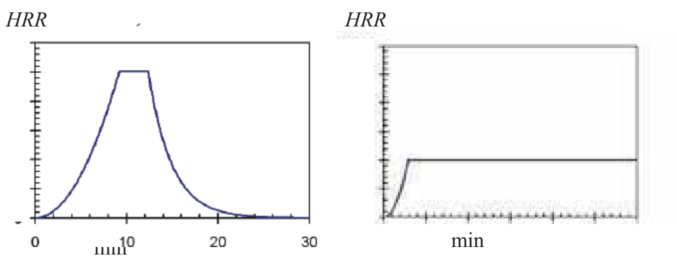



Fig. 10.5 Schematic fire loads (heat release rates, *HRR* in MW), local (left) and global (right) fires

The local and global fires in a sprinklered building defined above were assumed to occure at the most severe locations in the building.

The special uses, including abnormal uses, and corresponding fires were also assumed to occur in the building. The probabilities of these fire activations resulting from the size of the compartments and the occupancies are given in Tab. 10.1. The probabilities were calculated based on Tillander et al. (2009) for a 50 year period. Probabilities for abnormal uses were calculated assuming their occurrence once a month. The probabilities for local and global fires are given in Hietaniemi (2009).

Tab. 10.1 Probability of fire activations and sprinkler defects during 50 years of special uses of spaces.

| | First floor | | Second floor | Third floor | | Fourth floor |
|---|---|---|---|---|---|---|
| | Ice machine | Sto-rage | Coat-rack | Plastic slide | Stand (ab-normal) | Stage (ab-normal) |
| $A_r$ (m$^2$) | 4200 | 45 | 12 | 2000 | 260 | 900 |
| $A_r/A_{tot}$ | 0.221 | 0.002 | 0.0006 | 0.10 | 0.013 | 0.047 |
| Fire acti-vation | 6.8 E-01 | 1.2 E-02 | 3.2 E-03 | 4.2 E-01 | 5.8 E-03 | 2.0 E-02 |
| Sprinkler defect | 3.3 E-02 | 3.6 E-04 | 9.7 E-05 | 1.6 E-02 | 1.7 E-04 | 6.0 E-04 |

Next we shall consider the fire load intensities *q''* [MJ/m$^2$] for local and global fires.

Finnish regulations (Ympäristöministeriö (2002)) state that the value for stores should be more than 1200 MJ/m$^2$. For shops, exhibitions halls and libraries its proper range is 600-1200 MJ/m$^2$. For restaurants, smaller than 300 m$^2$ shops, offices, schools, sports halls, theatres, churches, and similar buildings the value is below 600 MJ/m$^2$. Based on the above, the maximum value for sporting areas is 600 MJ/m$^2$.

Measured data (International Fire Engineering Guidelines (2005)) yielded 421 fire load intensities for production spaces, which are clearly higher than in our cases. The mean of the sample was 530 MJ/m$^2$ and the deviation 540 MJ/m$^2$. The 3-parameter gamma distribution was used with the following results: 80 % fractile = 600 MJ/m$^2$ and 95 % fractile = 1100 MJ/m$^2$, see Fig. 10.6.

Based on these estimations, the following fire load intensities were used in this study:

– 600 MJ/m$^2$ for the spaces meant for sporting (80 % fractile for generic fire intensity distribution).
– 1100 MJ/m$^2$ for other spaces excluding stores (95 % fractile for generic fire load intensity distribution).

Next we shall consider the corresponding fire release rates (*HRRPUA*, Heat Release Rate Per Unit Area). Tab. 10.2 draws on data from Hietaniemi (2007). It presents the fire load intensities and corresponding fire release rates. The origin of each data line is given in Hietaniemi (2007).

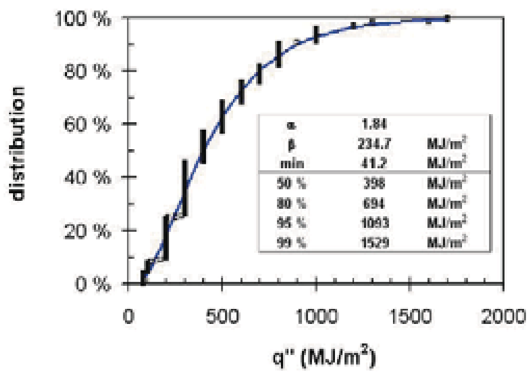



Fig. 10.6 Fire load intensity distribution for production space

Tab. 10.2 Sample of *HRRPUA* and *q''* values

| Item | $t_g$ (s) | HRRPUA ($kW/m^2$) | q'' ($MJ/m^2$) |
|---|---|---|---|
| Wood pile (4 pieces) | 209-409 | 469-2156 | 703-1561 |
| Stack of pallets (2 pieces) | 600-900 | 3062-4105 | 1500-2250 |
| One plastic chair | 900 | 600 | 160 |
| Stack of plastic chairs | 110 | 7600 | 1140 |
| Two stacks of plastic chairs | 110 | 4300 | 1450 |
| Sports bags | 420 | 1324 | 1829 |
| Fair stand | 150 | 1966 | 1203 |
| Litter basket (2 pieces) | 140-1450 | 1200-1400 | 400-422 |
| Carton | 150 | 1966 | 1230 |
| Work point in office (8 pieces) | 115-225 | 820-1799 | 376-914 |
| TV | | 930 | 500 |
| Washing machine | 273 | 1422 | 639 |
| Washing machine in cabinet | 563 | 1483 | 1054 |
| Refrigerator | 660 | 1921 | 1031 |
| Polyester coat | 720 | 250 | 40 |
| Coat-rack (2 pieces) | 150-210 | 188-190 | 90-125 |
| Shoe store | 80 | 2500 | 1760 |
| Speciality shop | 71 | 2900 | 2900 |
| Armchair | 120 | 5480 | 980 |
| Sofa (2 pieces) | 110-110 | 3120-3375 | 727-940 |
| Unprotected mattress | 145 | 527 | 126 |
| Protected mattress | 360 | 34 | 3 |

The close correlation between fire load intensity and heat release rate per unit area is shown by Fig. 10.7.

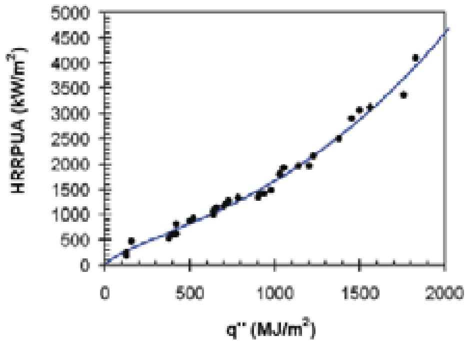



Fig.10.7 *HRRPUA* versus fire load intensity

Thus, fire release rates can be estimated based on fire load intensities:

- For sporting areas the mean fire release rate is 1000 kW/m$^2$ and its 5 % and 95 % fractiles are 800 kW/m$^2$ and 1100 kW/m$^2$ (see Fig. 10.8 a)).
- For other spaces (excluding stores) it is 1900 kW/m$^2$ and its 5 % and 95 % fractiles are 1600 kW/m$^2$ and 2100 kW/m$^2$ (see Fig.10.8 b)).

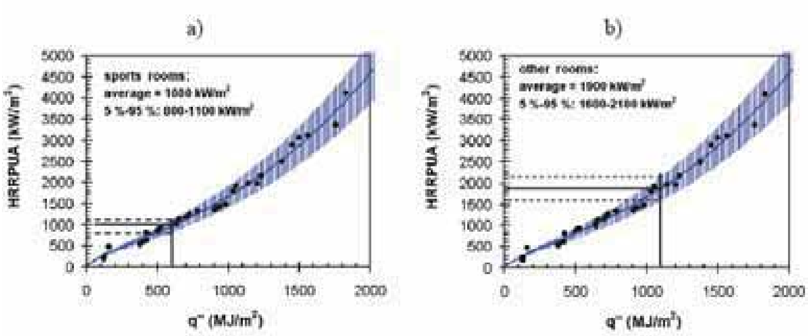


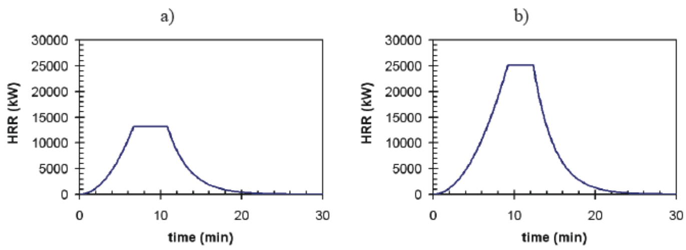

Fig. 10.8 *HRRPUA* for two types of spaces, shading indicates values between 5 % and 95 % fractiles

Next, we fit the local fire ($A_r$ = 12 m$^2$) curve of Figure 3 to the data above. In the growing phase we use the $t^2$ curve including the time $t_g$ needed to reach a heat release rate of 1 MW. In the decay phase we use an exponential curve including a creeping factor of 30 % of total heat release based on experimental values. Fire intensity is:

$$Q(t)=\begin{cases} Q_0\cdot\left(\dfrac{t}{t_g}\right)^2 & when \quad 0\le t\le t_1 \quad (growing\ phase) \\ Q_{\max} & when \quad t_1<t\le t_2 \\ Q_{\max}\cdot\exp(-\dfrac{t-t_2}{\tau}) & when \quad t>t_2 \quad (decay\ phase) \end{cases} \tag{1}$$

where $Q_0$ = 1 MW, $t_g$ = 150 s, $\tau$ is the creeping factor and $t_1$ and $t_2$ are the limit times for uniform fire intensity.

The result for the sports area is shown in Fig. 10.9 a) and for other areas in Fig.10.9 b). The maximum *HRR* for the sports area is little below 15 MW and for other spaces little over 25 MW.




| | | |
|---|---|---|
| fire area $A_f$ = | 12 | $m^2$ |
| HRRPUA = | 1100 | $kW/m^2$ |
| q' = | 600 | $MJ/m^2$ |
| max HRR = | 13200 | kW |
| Q = | 7200 | MJ |
| $t_g$ = | 110 | s |
| $t_1$ = | 400 | s |
| $t_2$ = | 650 | s |
| $\tau$ = | 160 | s |

| | | |
|---|---|---|
| fire area $A_f$ = | 12 | $m^2$ |
| HRRPUA = | 2100 | $kW/m^2$ |
| q' = | 1100 | $MJ/m^2$ |
| max HRR = | 25200 | kW |
| Q = | 13200 | MJ |
| $t_g$ = | 110 | s |
| $t_1$ = | 552 | s |
| $t_2$ = | 740 | s |
| $\tau$ = | 155 | s |

Fig. 10.9 Local design fires and their parameters

The *HRR* for the corresponding global fire is shown in Fig. 10.11.

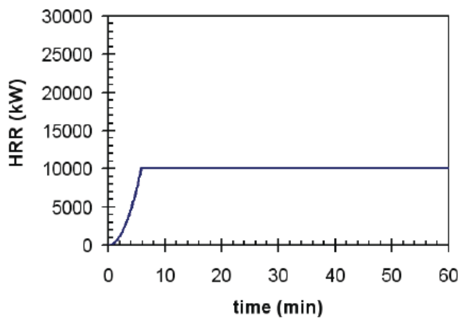



Fig. 10.11 Global design fire

In studying structures above the fire, the simplified geometrical model for modelling the local fire uses a square (3x4 = 12 m$^2$) at a specific height from the floor, and the fire burns only on the top surface. Special cases where the fire source was supposed to be 5 m above the floor were considered too.

The height $H_f$ of the fire source can be estimated using the equation:

$$H_f = \frac{q''}{\eta \cdot \Delta H_c \cdot \rho_{fuel}} \tag{2}$$

where *q''* is the fire load intensity, $\Delta H_c$ is the calorific value of the material (supposed to be within 25-44 MJ/kg), $\eta$ is the factor that accounts for the solidity of the material (one for a solid material, zero for a loose material) and $\rho_{fuel}$ is the density of the material (supposed to be within 900-1200 kg/m$^3$).

IF we suppose for simplicity uniform distribution of all the quantities within the ranges shown above and a 10-90 % range for the factor $\eta$. Then, based on 1000 Monte-Carlo simulations we find that for 600 MJ/m$^2$ the value $H_f$ is smaller than about 20 cm (Fig. 10.12 a) and for 1100 MJ/m$^2$ the value $H_f$ is smaller than about 50 cm (Fig. 10.12 b).

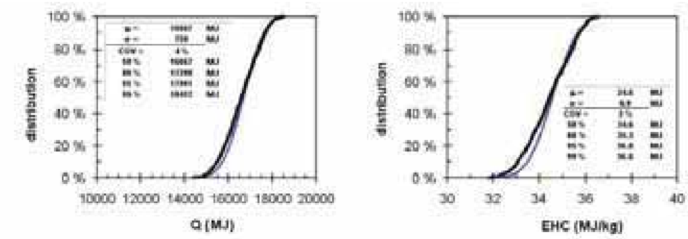



Fig. 10.12 Distributions for the burning item

Traditionally the value $H_f$ = 0.5 m is used for both cases. The fire source area used in the simulations is shown in Fig. 10.13.

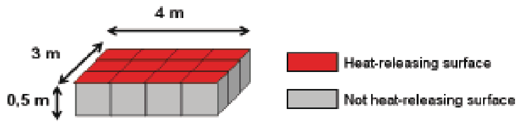



Fig. 10.13 Geometrical model for burning item (local fire)

Next, we shall consider the design fires for special uses.

<u>Ice resurfacing machine fire</u>

Two kinds of approaches were used to define the design fire for this case: simulation with the FDS 5 program and estimation with a general fire model for vehicles (Hietaniemi 2007). The goal was to define the design fire for the ICECAT (2008) machine shown in Fig. 10.14.


Fig. 10.14 Ice resurfacing machine

The machine contains the following combustible materials: plastics (ABS), glass-reinforced plastic (GRP) and rubber. The properties of ABS were derived from Lyon & Walters (2001) and Scudamore et al. (1991), those of GRP from Mouritz & Mathys (2006) and those of rubber from Iqbal et al. (2004), Chapter 7.

The machine was modelled with FDS 5 using cubes fit to the grid size and amount, distances, total size and mass of the cubes fit to the machine data.

The thermal properties used in the simulation were typical for plastics: density 1100 kg/m$^3$, thermal conductivity 0.2 WK$^{-1}$m$^{-1}$ and specific heat 1500 JK$^{-1}$kg$^{-1}$. Combustion time is estimated at 30 s and combustion temperature at 320 $^{\circ}$C. The fire is supposed to reach its maximum intensity in 60 s.

The simulation is based on normal distributed fire load [*Q*, MJ], effective net caloric value [*EHC*, MJ/kg] and heat release rate per unit area [*HRRPUA*, kW/m$^2$].

In the simulations the following 95 % fractiles were used as input. Their standard deviations are shown in parentheses:

– *Q*: 16700 (750) MJ,
– *EHC*: 35 (1) MJ/kg,
– *HRRPUA*: 700 (70) kW/m$^2$.

In some cases other fractiles were used to determine the effect of the input on the result. The result of the simulation is shown in Fig. 10.15.

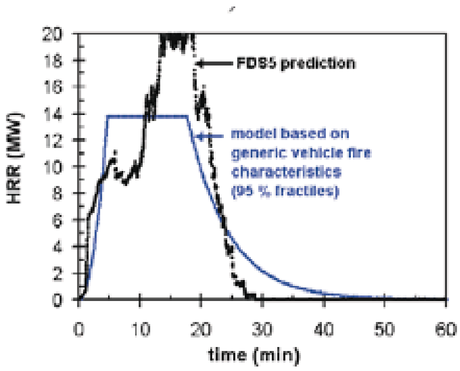



Fig. 10.15 FDS 5 prediction and general vehicle model prediction for the ice resurfacing machine fire

Fig. 10.15 also shows the result based on Hietaniemi (2007) using the 95 % fractiles 2225 MJ/m$^2$ for the fire load and 1725 kW/m$^2$ for the heat release rate.

The final design fires for the ice resurfacing machine were determined based on these analyses. They are presented in Fig. 10.16. Fig. 10.16 a) presents the local fire and Fig. 10.16 b) the global fire where after the total collapse of the sprinkler system the heat release rate doubles and then remains constant.

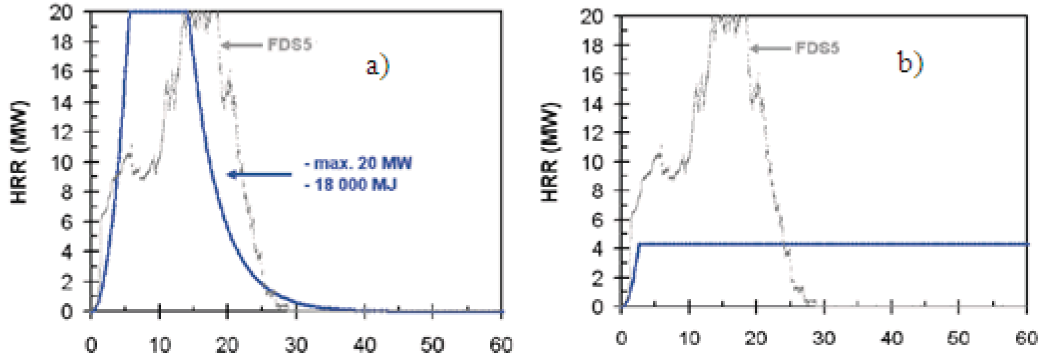



Fig. 10.16 Local and global ice machine design fires. Vertical axis shows time in minutes

<u>Storage fire with flashover</u>

The large compartment comprises storage spaces which should be divided into individual compartments using EI60 structures. However, the doors of the spaces open into the large compartment which is why the scenario where the door is open during the fire was chosen.

The storage space was modelled as a single sprinklered floor area because that represents the most severe situation as flames come out of the storage door. The fire load was modelled using 64 burning units each equalling a cell of the FDS grid. The heat release rate from each surface of each unit was 500 kW/m$^2$. The net caloric value was 35 MJ/kg and the total fire load 30,000 MJ. The FDS 5 model and an example of the flaming through the door are presented in Fig. 10.17.

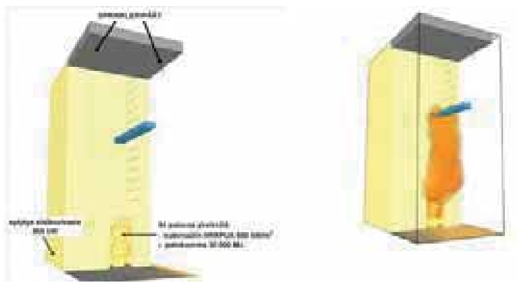


Fig. 10.17 FDS 5 model for predicting the storage fire load and an example of flaming through the door

The design fire for this case is shown in Fig. 10.18.

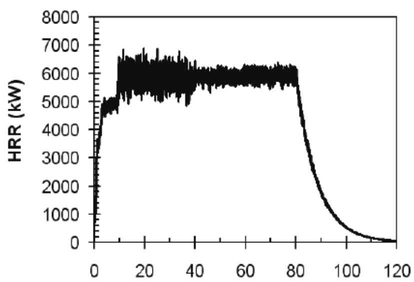



Fig. 10.18 Design fire for storage with flashover (time in minutes)

Coat-rack fire with local flashover

The definition of the design fire for this case started by modifying the FDS 5 model to simulate closely the experiments of Hadjisophocleous & Zalok (2004). The *HRRPUA* was 160 kW/m$^2$ and the *EHC* was 30 MJ/kg. The geometrical model, the FDS 5 model and examples of the fire are presented in Fig. 10.19.

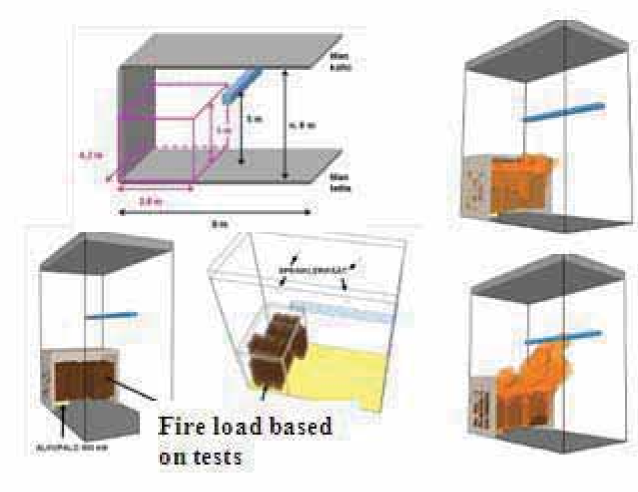



Fig. 10.19 Geometrical model, FDS 5 model and examples of fires to predict the coat-rack fire

The fire loads for the basic case and two variations are presented in Fig. 10.20. The first variation is calculated using double the *HRRPUA* [kW/m$^2$] value of the basic case. The second variation is calculated using double the fire load intensity [MJ/m$^2$] of the basic case.

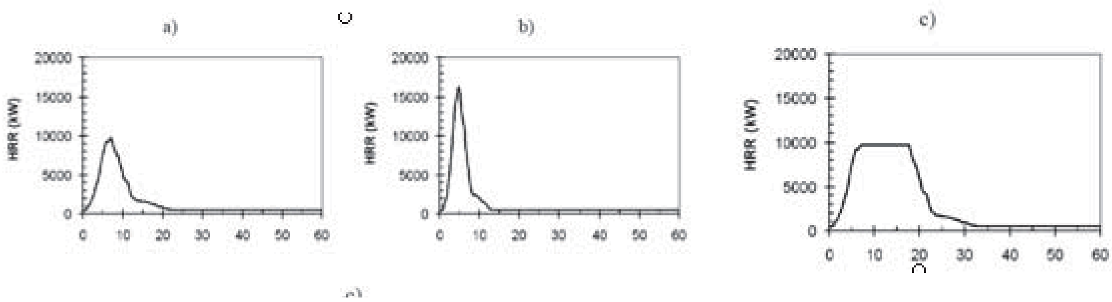



Fig. 10.21 The basic case (a) and two variations: doubled heat release rate (b), and doubled fire load intensity (c).

The fire of Fig. 10.21 c) was used in the final simulations of the building fires.

Plastic slide fire

The most hazardous object in the adventure space for children in case of fire is the plastic slide which is high and contains a lot of combustible materials. The slide and its simplified model are presented in Fig. 10.22.

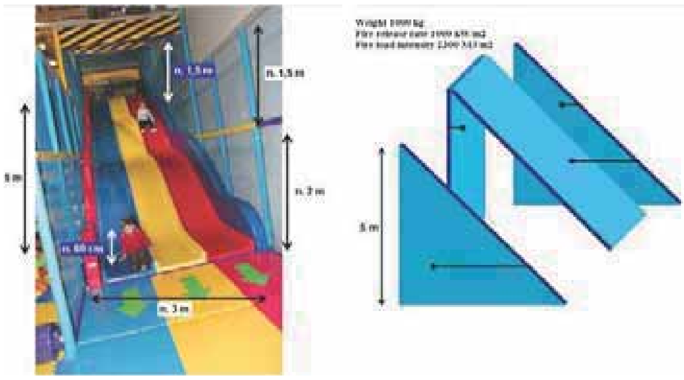



Fig. 10.22 Plastic slide and its simplified geometrical model

The design fire presented in Fig. 10.23 was used for global fire.

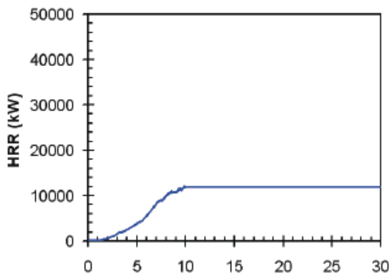



Fig. 10.23 Global design fire for plastic slide (time in minutes)

The design fire used in this case was much larger than the doubled fire load after sprinkler activation (about 5 minutes in Fig. 10.27).

<u>Stage fire</u>

The stage is not a permanent structure and is not normally in use. However, it may be needed in the dance, which is why this scenario was also considered. Stage load was defined for the area of one sprinkler (12 $m^2$). The geometrical representation of the stage and the quantity data for calculating the fire load are given in Fig. 10.24.

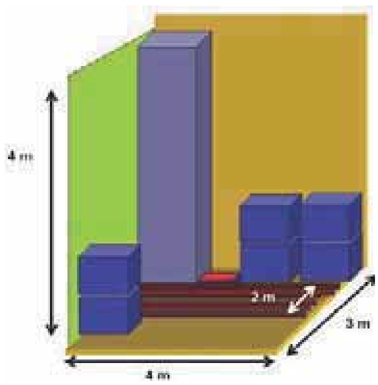



Fig. 10.24 Stage model

The quantity data and the corresponding fire are shown in Tab. 10.3.

Tab. 10.3 Quantity data of stage fire

| | Density [kg/m$^3$] | Heating value [MJ/kg] | HRR [kW/m$^2$] | V [m$^3$] | A [m$^2$] | Weight [kg] | Fire load [MJ] |
|---|---|---|---|---|---|---|---|
| Speaker | 200 | 30 | 1000 | 0.96 | 8.24 | 192 | 5780 |
| Amplifiers | 200 | 30 | 1000 | 0.86 | 8.40 | 173 | 5184 |
| Cables | 1200 | 40 | 450 | 0.72 | 60.84 | 144 | 5760 |
| Platform | 700 | 15 | 1000 | 0.30 | 12.35 | 60 | 900 |
| Back wall | 700 | 15 | 1000 | 0.40 | 16.40 | 80 | 1200 |
| Curtain | 1200 | 40 | 1000 | 0.01 | 12.01 | 2 | 96 |
| Total | | | | | | | 18900 |

The attributes of the single homogeneously burning stage material for the whole area are:

HRRPUA = 1000.0 kW/m$^2$

THICKNESS = 0.05 m

DENSITY = 1200.0 kg/m$^3$

HEAT_OF_COMBUSTION = 30.0 MJ/kg

The fire load of the stage is presented in Fig. 10.25 with the fire load of a global fire (red line).

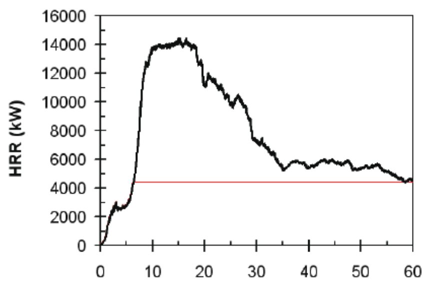



Fig. 10.25 Stage design fire loads, local and global (red)

Stand fire

The stand is not a permanent structure. Temporary stands are needed for spectators of beach volley and badminton matches. The stand is made of plywood and plastics. Its geometrical model is given in Fig. 10.26. The quantity data and corresponding fire load calculations are shown in Tab. 10.4.

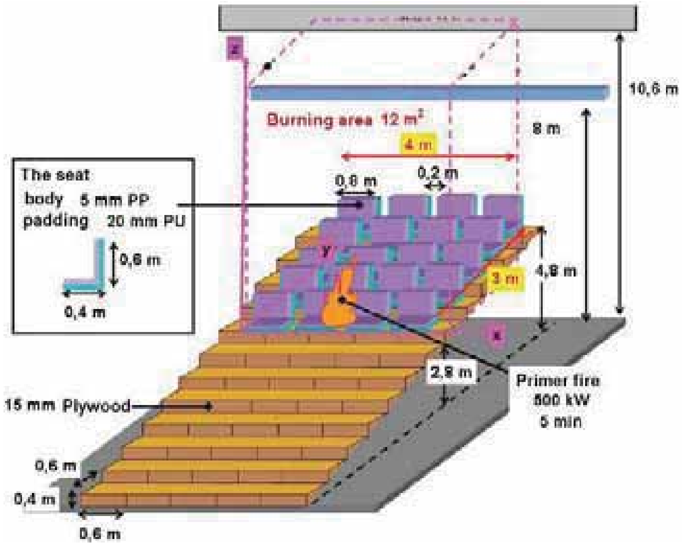



Fig. 10.26 Geometrical fire model of the stand

Tab.10.4 Quantity data of one seat in the stand

| | Density [kg/m$^3$] | Heat value [MJ/kg] | HRR [kW/m$^2$] | V [m$^3$] | A [m$^2$] | Weight [kg] | Fire load [MJ] |
|---|---|---|---|---|---|---|---|
| Plywood | 700 | 15 | 150 | 0.0072 | 0.72 | 5.04 | 75.6 |
| PP | 1200 | 40 | 1200 | 0.0024 | 0.72 | 2.88 | 115.2 |
| PU | 100 | 25 | 400 | 0.0096 | 0.72 | 0.96 | 24.0 |
| Total | | | | | | | 214.8 |

The size of the burning area is 12 m$^2$. The attributes of the single homogeneously burning stand material are:

HRRPUA = 583.0 kW/m$^2$

THICKNESS = 0.2 m

DENSITY= 52.0 kg/m$^3$

HEAT_OF_COMBUSTION = 24.2 MJ/kg

Local and global design fires for this case are given in Fig. 10.27.

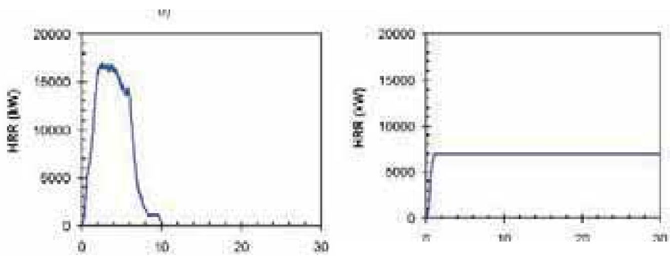



Fig. 10.27 Local and global design fires for the (time in minutes)

### 10.2.3 Estimations of errors in results

Some error estimations concerning the proposed design fires should be done before any fire simulations on the building. The selected fire scenarios meet the requirements of Finnish regulations (Ympäristöministeriö (2002), Chapter 1.3.2) and, thus, cover all fires that probably could take place in the building. They do not represent the average situation, but a rare situation which can be considered to represent 99 % of the cases. This means that one fire out of 100 can be worse than expected. That is a very small number, which means that in this study the possible uncertainty of the fire scenarios will be attributed to the uncertainty of the design fires.

The uncertainty of design fires consists of the uncertainty of our knowledge and our ignorance (epistemic and aleatoric uncertainty) such as:

- The values used in calculations, e.g. *HRRPUA* values, always include noise originating from non-ideal tests arrangements, measurements and analysis models.
- Possible systematic errors in the values used in calculations originating e.g. from the hypotheses made to simulate the real situation.

The uncertainty of fire technical measurements is of the order of 20 % as are model uncertainties. Assuming that systematic uncertainties are of the same order (20 %), the uncertainty $\Delta\dot{Q}$ of the fire load is

$$\Delta\dot{Q} \approx \sqrt{20\%^2 + 20\%^2 + 20\%^2} \approx 34\% = \pm 17\% \qquad (3)$$

According to fire plume models, gas temperature $T_g$ rises in proportion to ambient temperature to the power 2/3 as shown by Heskestad (1984) and Hostikka (1997).

$$T_g \propto \dot{Q}^{2/3} \qquad (4)$$

so the uncertainty $\Delta T_g$ of the temperature rise is

$$\Delta T_g \propto \frac{2}{3}\dot{Q} \qquad (5)$$

This means that the relative uncertainty of the estimations of temperatures can be described as a normal distribution with a mean of 1 and a standard deviation of 10 %:

$$\frac{\Delta T_g}{T_g} \propto N(1;10\%) \qquad (6)$$

### 10.2.4 Fire simulations

<u>The simulation environment</u>

The aim of the simulation was to estimate endurance of structures to natural fire. The structural product model was used as the basis of simulation. Beams, columns, roof and floor slabs, and concrete stairwells were incorporated in the model. The data content of the structures of that model was more complete than that of the architectural model. The building parts were not assumed to be involved the fire since all the burning material was assumed to be included in the fire packages. The material properties of structures were not needed in fire simulation.

The structural model was complemented based on drawings. The airspace where the fire burned was bounded by slabs or wall panels. All doors were modelled as openings in the walls assuming that evacuated persons had left them open. Other vents were for the most part not modelled. If there were any openings, the airspace where the fire burned could also be modelled by the properties of the edge of the calculation grid. The used modelling program was Tekla Structures version 15.0.

The NIST Fire Dynamics Simulator (FDS) version 5.2.5 was used for simulation. The calculation method is based on CFD (computational fluid dynamics) which uses a three- dimensional, rectilinear computation grid. All the modelled objects must be modified into cubes in some phase of the data transformation process.

A special data transformation program was used to transfer the structural model data to the FDS input file. At the same time, all needed material data were stored to the same input file. The process is described more accurately by Laasonen (2010).

Selection of the grid cell size

The size of a single cell of the calculation grid affects the following three important factors given in order of importance: 1) the reliability of simulation, 2) the minimum size of the objects that can be incorporate in the fire model, and 3) the computer time needed for calculations.

Heinisuo et al. (2008a) have discussed the required cell size. Heskestads's correlation is used to estimate the reliability of calculation. It uses the density of fire [kW/m$^2$] and the burning area to calculate the so-called Resolution factor (R) for defining the sizes of cells. Heinisuo et al. (2008a) recommended that the sizes of cells should be selected so that the value of R is at least 10 (or inverse value r not more than 0.07).

As presented in the previous chapters, the used special fires are not planar but involve three-dimensional objects which may burn on many faces. Then, the acceptable limit for the Resolution factor is not known. Two Resolution factors have been calculated based on simulated fires: a lower value when only the fire on the top face is included in the burning area, and the higher value when all the burning faces are included in the area.

To limit calculation time, the model was divided into the several grids. A calculation environment where every grid can be calculated by a different processor was used. However, the hottest area was not divided between several grids because that could cause problems to the stability of calculation. Also, if a larger number of processors are needed, the starting of calculations could be severely delayed.

Coarser grids were used for the colder parts. Alpert's correlation was used to approximate the width of the hot area. A distance from the plume centreline where the temperature should be less than 100 ˚C was calculated. This distance is always smaller than the distance to the edge of the coarse grid.

In the simulation environment the co-ordinates of modelled objects were not changed in the transformation to the fire simulation program. The simulation program was allowed to locate every co-ordinate to the nearest cell corner using normal mathematical rounding rules. If all the corners of an object are rounded to the same cell corner, it will vanish from the fire simulation. Because of rounding, the thickness of some objects may be zero. As long as the rounding cause any unwanted holes in the simulation model, it should not affect the calculation. The simulation program reads the real thickness of objects from their attributes.

The effect of rounding was observed by two methods. In the simulation environment the calculation grids were also added to the structural model. At least one edge of the grid could be located according to modelled structures. All the added geometry could also be located to the grid cells. For example, holes less than two cells in size were not used.

The other method involved visual checking of the fire simulation model. The checking was carefully done before calculation when most of the problems could been noticed. After calculation, smoke animation could indicate unwanted air flows.

To minimise calculation time, the biggest possible cell size was usually selected. Then the rounding of co-ordinates may cause structures to be lost in the fire simulation model. Profiles whose both dimensions are less than the cell size will probably be lost if not successfully located between cell corners. Profiles exactly the size of a cell can be lost if the cell corner is located exactly in the middle of the profile. That is highly improbable.

Heinisuo et al. (2008a) have tested the effect of different sizes of obstacles in a fire model. They noticed that if the obstacle height versus corridor height is below 0.1 in a ceilinged space, and the obstacles are not located close to each other (less than three times their height), it is not essential to model them in a fire simulation. Consequently, slender profiles do not change substantially the flow of air. The height of the modelled spaces was typically between 4 and 10 metres. Then it can be assumed that ignoring of obstacles smaller than 400 mm has little effect on simulation.

In the hot area the upper limit of cell size was 200 mm. Outside the hot area, the flow of air is even slower and bigger obstacles can be ignored in the fire model. There the upper limit of the cell size was 400 mm. The end result of the investigation of the effects of rounding was that profiles smaller than the cell size could be freely rounded off. The pictures of the fires in Chapter 5.5 show that, for example, all diagonal members of trusses have vanished from the fire models.

Modelling of fires and grids

The previously presented fire packages were used in simulations. The properties and behaviour of burning materials were converted to FDS language. The HRRPUA, CONDUCTIVITY, SPECIFIC_HEAT, HEAT_OF_COMBUSTION and DENSITY values were given. The slope depicting the development of the fire as a function of time was given. The material data of the fire were linked to the model so that the name of the FDS fire was included in the name of the geometrical object describing the fire.

The fire was modelled in the form of cubic geometry which follows the cells of the calculation grid. The location of the fire was selected for maximal temperatures of structures. Then the flames should reach the structure or just underneath. The other rule was that there should be enough air for the fire since the area around the opening is the severest.

The finest grid was located around the fire. One edge of the grid was aligned with the bearing structures. The exact location of the fire was fine-tuned accordingly. Then the other fine grids where located around the first one. Finally, the rest of the model was filled by coarser grids.

Output of temperatures

Air temperatures were output at certain points during fire simulation. The location of the points must be entered by co-ordinates to the input file of fire simulation. The middle point of every steel member was selected as a control point. That allowed reading the co-ordinates automatically from the structural model. Temperatures at different locations of long and vertical rods varied sometimes. The safe solution in such instances is to assign critical members the highest calculated temperature of the surroundings. In some cases extra control points above the fire were also included in the calculation.

The air flow near the flames and plumes is turbulent. The programs can simulate this when output temperatures vary a lot between successive calculation steps. In an intense fire the difference could be about 100 °C. If we wish to know the temperature at one point at a certain time, it is not advisable to take a single value from the time-temperature curve because of the turbulence. It is better to use the so-called 'sliding window' with the mean of several successive calculation steps.

One simulated second may involve several steps of calculations. That would make the amount of output data huge. The temperature of structures corresponds closely to the temperature of air. For these reasons, all the calculated steps are not used in post processing. Hostikka et al (2001) have presented an equation to calculate the width of the sliding window. In the output diagrams of simulations they reduced air temperatures to 10 seconds wide time steps. That value was considered suitable in all cases.

The temperature of a steel part can be calculated by integration from the time-temperature curve of air. Heinisuo et al. (2008b), among others, have presented examples of such calculations.

In the following, only the air temperature curves are given. These temperatures were used by the structural engineers of the project to check the resistance of the structures in fire.

Simulated cases

The calculations of the fire cases presented in Chapter 3 were done to determine the worst-case scenarios. The cases involving the highest temperatures are presented in the following. Results are presented mainly for those control points where air temperatures were over 400 °C. That is a critical limit because the yield stress of steel decreases at temperatures above it.

Tab. 10.5 lists the documented cases. The Resolution factor (R) is output as told in Chapter 5.2. An exception is the coat-rack fire where the relative area of the top faces was very small and the top of the coat-rack was closed as shown in Fig. 10.34. The R value of the top faces in the coat-rack fire has not been output. All calculated values are at least near the minimum target value 10. The worst R value was calculated for the storage fire, but there only the top faces of the fire elements were burning.

The number of grids of both used cell sizes is given. The total number of grids of the fire models was between 7 and 16.

Tab.10.5 Documented simulations

| | Resolution factor R | | Number of grids | | |
|---|---|---|---|---|---|
| | Burning faces | | Size of cells [mm] | | Σ |
| | top | all | 200 | 400 | |
| Ice hall, ice machine | 9.6 | 13.9 | 3 | 6 | 9 |
| Ice hall, storages | 9.2 | - | 4 | 6 | 10 |
| Restaurant, coat-rack | - | 11.4 | 4 | 5 | 9 |
| Fun park, slide | 13.9 | 30.6 | 6 | 10 | 16 |
| Dance hall, stage | 13 | 23.8 | 6 | 2 | 8 |
| Volleyball hall, stand | 10.4 | 12.8 | 7 | 0 | 7 |
| Climbing hall, climbing wall | 13.4 | 25.6 | 8 | 0 | 8 |

The initial simulation time was one hour. In cases where the combustible material burned away, the simulation was stopped earlier. The output temperatures should have settled down before the stopping.

Tab. 10.6 shows the calculation times of simulations. The maximum numbers of cells in one grid and simulation time were output to compare different cases. As stated earlier, it is advisable to avoid dividing the grids around the fire to keep calculation times short. A long, intense fire also lengthens the calculation time in addition to the wideness of the grids.

Tab. 10.6 The simulation and the calculation times

| | Maximum number of cells of grids | Simulation time | Calculation time of simulation |
|---|---|---|---|
| | | min | hh:mm |
| Ice hall, ice machine | 109824 | 33 | 33:47 |
| Ice hall, storages | 109824 | 60 | 41:02 |
| Restaurant, coat-rack | 100000 | 60 | 33:51 |
| Fun park, slide | 83200 | 60 | 41:01 |
| Dance hall, stage | 72000 | 50 | 86:13 |
| Volleyball hall, stand | 52000 | 25 | 16:33 |
| Climbing hall, climbing wall | 190256 | 23 | 55:28 |

The ice hall was modelled in actual size bounded by the designed walls. The space was so large that the fire qualities of the walls did not matter in the simulation. The fire was situated near a door so as to provide enough air. The burning part of the machine was at the actual level.

Fig. 10.27 is an example of the visualisation of simulation. The door openings are white and the green points indicate where temperatures were output. Only a few bottom flanges of the trusses were included in the fire model while all other parts were rounded off.

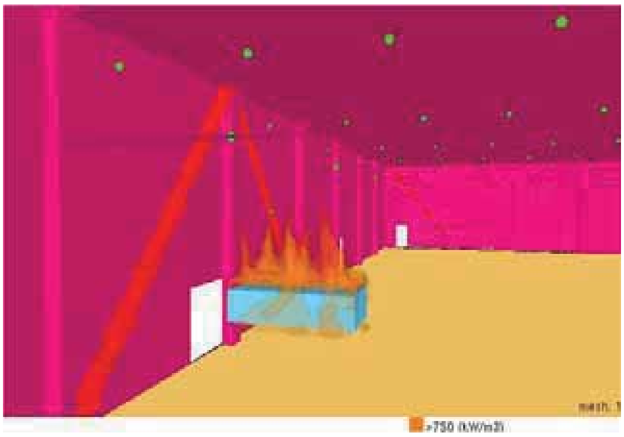


Fig. 10.27 The ice resurfacing machine fire

The time-temperature curve of Fig. 10.28 shows that the fire was decaying rather quickly. The control points are indicated by the letter 'B' followed by the consecutive number of the corresponding member.

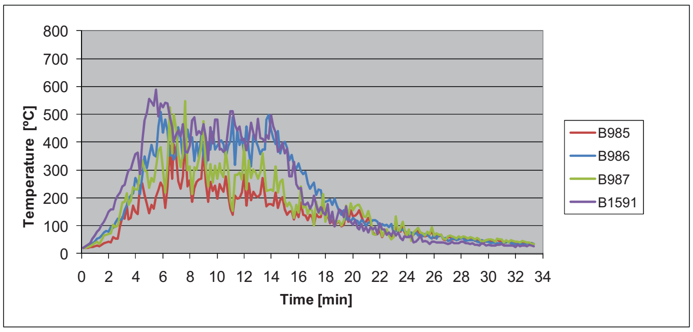



Fig. 10.28 The highest air temperatures above the ice resurfacing machine fire at various control points

In the case of the storage fire, burning objects filled the space and flames shot out of the open door. The structures most endangered by the fire were those above the door opening. In Fig. 10.29 air temperatures are represented by a coloured slice. The other colours of the slice only visualise temperatures while the red objects are structures. The figure shows that the ceiling above the door spreads the heat so that the air at the ceiling level is not very hot. On the other hand, the temperatures at the platform just above the door and the column are rather high.

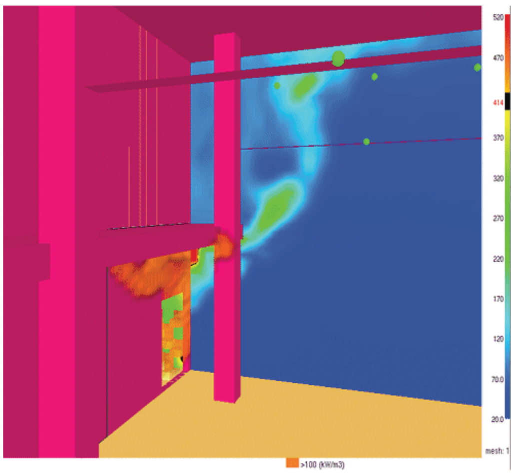


Fig. 10.29 Storage fire in the ice hall

The restaurant was modelled in actual size as an open space with all the doors open. Thus, the lack of air did not limit the fire. The burning coat-rack was situated according to architectural drawings. The fine tuning of its position was done by testing when the flames reached the bottom flange of a truss. The fire is visualised in Fig. 10.30. Air temperatures at control points were not raised above 400 °C.

Fig. 10.30 Coat-rack fire in the restaurant

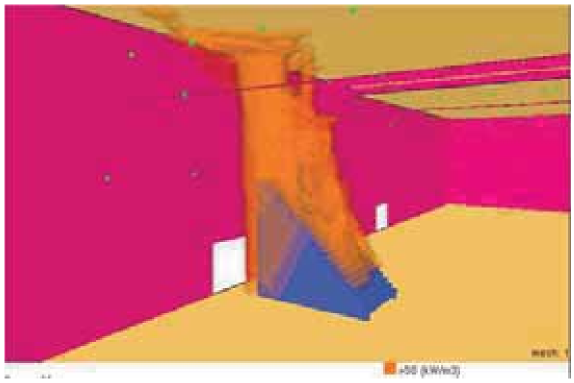


Fig. 10.31 The plastic slide fire

The plastic slide fire was situated in an open hall near an open door. No other equipment or possible separating walls were modelled. The fire is visualised in Fig. 10.31.

The plastic slide fire was very severe although the structure of the slide was thin. Thus, the combustible material was consumed quite quickly as shown by the time-temperature curve in Fig. 10.32.

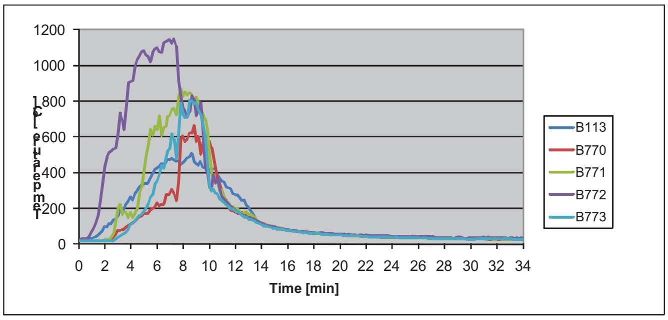



Fig. 10.32 The highest air temperatures above the slide fire at control points

In the dance hall model, the entire space was left open. The separating walls were left out of the model in order to produce simulation results on the safe side as the lack of air could not limit the fire. The fire was located near the emergency exit which was modelled open.

Test calculations were made to determine the most severe situation of the stage fire. It was noticed that that tallest speaker caused the highest and longest-standing flames. At the time, the speaker was located under the truss.

Fig. 10.33 shows the geometrical model of the storey where the volleyball hall is located –without the walls. The calculation grids can be seen as darkened areas on the floor. The thinner grids are indicated by the darkest colour. The perimeters of the grids follow the walls of the hall. The gray doors and ventilation opening are also pictured.

According to Chapter 3.8, only the upper part of the stand is assumed to burn. Therefore, only the seats of the upper part of the stand are modelled.

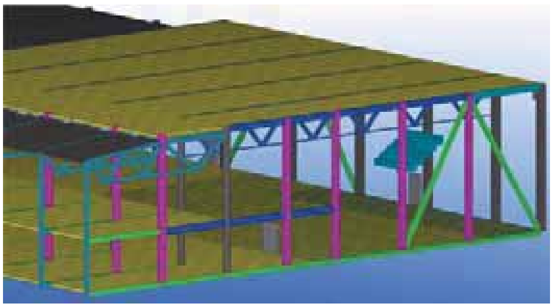


Fig. 10.33 The geometrical model of the volleyball hall

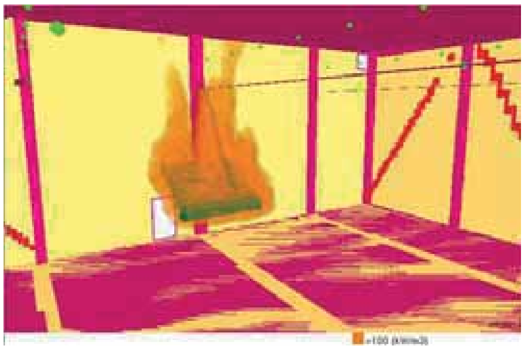


Fig. 10.34 The stand fire in the volleyball hall

The highest point of the flames varied across the stand. The locations of the highest temperatures varied correspondingly. Thus, it was difficult to determine the single most critical spot of the fire. Therefore, the highest registered temperature should be used for all structures above the fire. Fig. 10.34 also shows how the diagonal braces modelled in green in Fig. 10.33 have been modified into cubes in the fire model.

The air temperatures near the structures were high because the fire spread up towards the ceiling. The duration of the fire was, again, short as can be seen from Fig. 10.35.

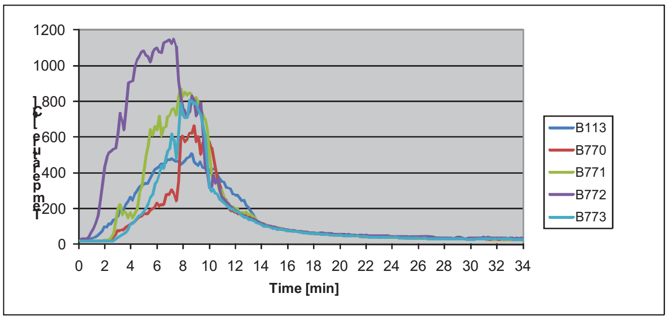



Fig. 10.35 The highest air temperatures above the stand fire at control points

The climbing hall was modelled as a three-dimensional multistorey space. Three trusses supported the ceilings. The climbing equipment and plywood based climbing wall were assumed to catch fire. A temperature slice was output also for the climbing wall fire of Fig. 10.36.

All simulation data were delivered to the structural engineer of the project. That allowed him to visualise the simulation results using all temperature histories of all control points. Using this information he could check the resistances of the trusses of every store.

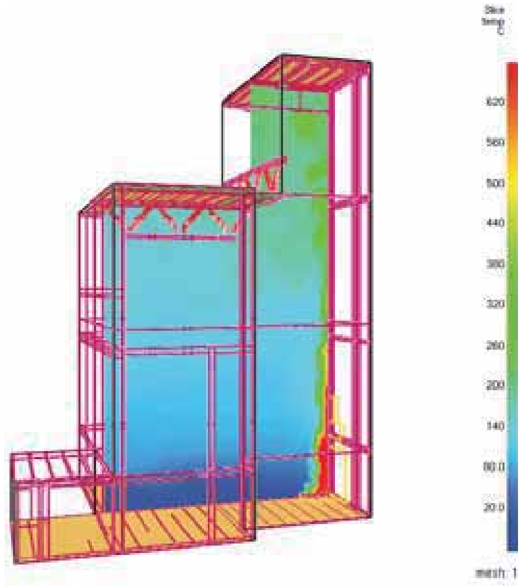



Fig. 10.36 The climbing hall fire

## 10.3 CONCLUSION

Performance based fire engineering is increasingly used in projects not only for evacuation, smoke control and exit design, but also to determine the resistance of structures in fire. It is not used just to minimise or reduce fire protection, but to enhance the fire safety of structures. In some cases it provides better fire protection than traditional fire design.

Performance based fire design is not suitable only for large projects, but for all projects

Lot of work will be required in Europe to bring fire design to the same level in different countries, which would make the market for products subject to the same regulations wider. Fire design has been typically incorporated in different sections of national codes as structural codes. In many countries national rules have been changed to allow applying Eurocodes to fire design as required in EU regulations.

The lack of experience and confidence of authorities and design fire definitions seem to be the largest challenges to performance based fire design in projects. The checking of design calculations is a major challenge to authorities.

The article presented one simple case study using water extinquishers and a case study on how to define the temperatures of fire compartments. Only the fire scenarios and the definitions of the design fires were given. The structural design of the project was done by others.

Fire scenarios for all the parts of the building were defined in close co-operation with the client, the authorities and other designers of the project. In this kind of performance based design co-operation between all partners to the project is essential and leads to a thorough survey of the worst-case scenarios. The authors believe that the end result is a very high level of fire safety for buildings.

This kind of design requires first rate fire engineering skills and good computing facilities. The developed integrated fire engineering tool was used in the project. In this case a module was used to transfer the data between the product model (Tekla Structures) and the fire simulator (FDS). Careful grid sizing, fitting the obstacles and fire packages to the right locations, etc. require experience from the end user of the system.

Similar integrated systems, in fact the same simulator, FDS, can be used e.g. in evacuation design and other design tasks. High unused potential lies in the integration of design procedures. However, the expertise of skillful engineers cannot be substituted by computers.

Performance based design should be incorporated in design at an early stage of the project. In the case study it was done by the steel contractor at a rather late stage. Earlier introduction could result in improved fire safety over the life cycle and bigger savings during the building phase compared to this project.

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# 11 FIRE IN A LARGE-AREA SHOPPING CENTER

Summary

The paper presents an analysis of a fire in a large-area shopping center. Although the building was equipped with fire protection measures, their activation has not proved to be effective. The fire took place at night without need to evacuate customers. It was found that the direct cause of the fire flashover was short circuit in the electrical wiring and improper use of the shelving containing parts made of combustible materials. In this report, the authors suggest, however, many indirect causes, needlessly generating significant fire hazard. They are characteristic for the whole group of such facilities. The primary risk factors included, among others are: the applied fire protection system which was satisfactory from the legal point of view but not uniform throughout the building, defective, and easygoing manner of use and lack of professional training of personnel. In final remarks, there are given recommendations to investors and managers of large shopping centers on the design and implementation of new structures and on maintenance of the fire protection systems providing required level of safety in buildings already in use.

## 11.1 INTRODUCTION

In the first half of the nineties of the last century a new category of buildings - large shopping centers emerged in Poland as a side effect of the political and social changes. Initially, these were mainly single-story buildings, in later years they were dominated by multi story malls. These objects relatively quickly replaced the traditional department stores where the floor area did not exceed several thousand square meters. The area characterizing the largest of the newly created centers is now almost fifty times bigger. They are, therefore, the places where thousands of people, employees and customers are gathered in a limited space, in the midst of a huge range of goods. The specific usage carries the challenge of providing the necessary level of security through developing appropriate rules of protection against fire flashover. However, this cannot be a simple adaptation of the general rules, applicable generally in the typical public buildings.

The need for refinement and continuous improvement of a variety of specific recommendations, that take into account all possible scenarios and fire hazards, was indeed noticed relatively quickly, especially in the case of the considered shopping centers. A catalyzing factor in this field was a series of fires that have occurred in such facilities in recent years. At the beginning the applied solutions for fire protection were yet consistent with the law in force at that time, however, they were incoherent with the modern safety requirements. An investor not obliged to use relatively expensive solutions generally choose those being inexpensive and ineffective if only they satisfied the current low requirements. An example was a legal rule allowing for refraining from installing fire alarm system if the object was protected only by permanent firefighting equipment. Foreign investors were also surprised by no obligation to separate fire compartments for storage facilities. Considered objects at the time of use were modernized several times. The way the commercial space was used also changed, mostly due to acquisitions by new tenants. Any such change was associated with other safety requirements, effective at the time of modernization. As a result, today we have complex objects with parts not fully congruent to each other and with quite different levels of fire protection. In many cases, spaces used in a similar way have different limitations, for example regarding the maximum height of storage or kind of goods accepted for the trade.

Published recently in Poland a set of rules recommended for operation in this field (Skaźnik, 1999) is the practical help, particularly useful for investors, managers and users of large retail stores, but also for all interested on requirements, systems and procedures for fire protection of buildings of this type.

## 11.2 BUILDING DESCRIPTION

An example of the facilities described above is a large-area shopping center where the fire took place in December 2008. It was built and handed over for use in the late autumn of 1999. The total building area was approximately of 48 000 m$^2$, and the usable area was about 50 000 m$^2$. The whole building was divided into three main fire compartments of the areas 10 400. m$^2$, 22 000 m$^2$, and 14 200 m$^2$, respectively. In the zones II and III the main tenants occupied from 2/3 to 3/4 of the total area, which consisted of sales room and warehouse facilities, separated by fire-break divisions with the increased fire resistance. The remaining area of each compartment was occupied by a strip of the retail outlets and the other service facilities. Additionally, the separate fire compartments were designated for the sprinkler central units and for some technical rooms. In May 2005 the fire compartment III was expanded by 1 600 m$^2$. Finally at that time the sales room was an area close to 9 000 m$^2$ and the largest warehouse facilities in this part had 700 m$^2$ and 1 300 m$^2$.

The superstructure of the considered object was represented by reinforced concrete columns, steel beams and steel purlins. The curtain walls to a height of 3.5 m were made of prefabricated elements, and above that height - as a layered wall, consisting of two layers of sheet steel, internally insulated with mineral wool. Interior walls were made of brick or plaster - cardboard. The roof was made of corrugated steel sheeting, insulated from the top with self-extinguishing polystyrene, sealed with vapour barrier foil on both sides and coated on the outside with topcoat coverage foil. The entire building roof was classified to the Class E of fire resistance (which means no special R, E, or I reqirements according to Rozp., 2002) with particular elements belonging to the non-spreading fire category. On the other side, two-story part of the building structure was classified to the class D, more restricted in relation to the required fire resistance. Furthermore, the whole storage zone separated in considered shopping center was qualified also to the class E. In addition, all members and all materials applied in its structure could not spread fire. Finally, the elements separating fire compartments had fire resistance of at least 120 minutes.

**11.3 EXISTING FIRE PROTECTION SYSTEM**

Before the enlargement of the building took place, it was equipped with all the fire equipment required by the contemporary law regulations (Rozp., 2002, Rozp., 2010), including:

- protection sprinklers covering the entire space of the shopping center, excluding such facilities as the electrical switchgears, monitoring rooms, air units, sprinkler systems, refrigerators, etc.
- alarm-signaling installation, consisting of the smoke detectors built-up in the passage and the spaces adjacent to commercial premises - services (excluding the sales floor), the manual fire alarms located in the entire facility, the fire control central connected directly to the city fire department,
- smoke extraction system, covering the entire site, including storage rooms, and consisting of the windows and smoke vents, pneumatically and electrically operated. The smoke extraction at the passage was secured by the system of windows operated by the signal from the fire central, upon detection of smoke by fire detectors. The passage space under roof was divided into 11 sectors by smoke curtains with the supply of fresh air in the passage provided by the escape sliding doors, automatically opened in case of fire, and revolving doors operated manually by the staff, the selling rooms were equipped with smoke dampers operated automatically or remotely from a set of points distributed in several places at the object,
- battery powered emergency lighting central.

After the expansion in 2006, the following changes were applied:

- in the refurbished store new sprinklers designed as a single level, supplied from the existing sprinkler pump, were installed with the resulting intensity of the spray 15 mm/min,
- fire alarm system was installed in the main hall of the sale, the linear smoke detectors were used with the time delay between the discovery of fire and switching on the central fire alarm of the second degree set at 3 min.,
- the extended part of the store was equipped with a device containing of the smoke vents controlled by the fire alarm system.

The range of the exhibited goods in the main hall included goods with very different susceptibility to inflammation, since made of metal, through wood and plastic up to the acrylic enamels and solvents. In the electrical section on the shelves there were exhibited among others wire coiled on spools made of plastic and wood.

### 11.4 COURSE OF FIRE

The course of fire has been reconstructed on the basis of information recorded in the fire control unit, at the alarm receiving center and fire monitoring system, and based on the analysis of footage from the cameras of the object. It looked as follows:

- Thirteen minutes before midnight, the lights go out at the racks in the electric department,
- After 10 seconds in the view of one of the cameras flame appears at the top of one of the shelves in the electrical department - This point was taken as time 0,
- 2.5 minutes from time 0 – there are visible flaming droplets sinking on the lowest part of the shelf, where there is a second outbreak of fire,
- 8 minutes - the first fire detector signals the fire, and immediately after, the next one, both alarms are deleted by the shop service,
- 9 minutes – the fire covers the second portion of the rack, at the time when the third detector signals the fire alarm, also cleared by the shop service,
- 10 minutes - the fire spreads over the full height of the whole wall of the unit, and at the same time the sprinkler is opened, causing visible suppression of the fire and smoke rise, at the same time the fire appears on the other side of the rack on the floor, the opening of the sprinkler transmits the alarm to the municipal fire brigade,
- 10 minutes 20 seconds - the fire covers two segments to the entire height of the rack, followed by rapid development of fire - probably due to ignition of the insulation of cables stored in that location,
- 13 minutes - the sprinkler built across the shelf gets on but the fire is still rapidly evolving,
- 17 minutes - the first units of the state fire service arrives,
- 18 minutes - the fire is spreading over the roof of the object.
- Extinguishing action ends after 108 hours of fire brigade activity. As a result of the fire almost the entire fire zone of several thousand square meters was destroyed. The fire also spread to the back warehouse store. The extent of the damage in shown in the photography (Fig. 11.1).

Fig. 11.1 General view of burned commercial building

**11.5 DIRECT CAUSE OF FIRE**

The preliminary analysis showed that the probable cause of the fire could be overheating of so called couplings used in electrical connections supplying lighting fixtures for shelving. Under normal lighting conditions, it should be turned off after trading hours, however that day, for unexplained reasons there was no such exclusion. The spread of fire was possible due to the specific design of the racks. Across the hall the selling goods were exposed on the shelves made of non-combustible materials. These racks were combined in bilateral rows, and the rear walls were made of perforated sheet metal. However, in some areas of the room, particularly in the electrical department where the fire broke out, there were additional wood panels made of combustible material to which in turn some electrical equipment and lighting fixtures were mounted.

**11.6 FACTORS GENERATING FIRE RISK**

**11.6.1 Consistent with legal requirements but nonuniform fire protection**

As mentioned above, the facility was put into operation in 1999. The fire protection system used at that time satisfied all the requirements of the current building law (Rozp., 2002 - in the previous version from 1994, Rozp. 2010). In 2006 the object was extended. In the new part of the building more modern and efficient protective devices were installed, in accordance with already more restrictive law (Rozp., 2002, and CEA, 2003). Thus, there was a situation where in one room there were sprinkler sections designed according to different standards, and imposing different conditions for storage of goods. Moreover, according to the project documentation, all the smoke vents in the new part of the store should be controlled by fire alarm system. Finally, as a result of savings, much of the vents were operated by individual thermal triggers. It should be also mentioned that the fire protection project made in 1999 did not set out in principle any conditions for storage of goods. Such requirements were given only in 2006 and only for the modernized part of a building.

#### 11.6.2 Faulty operation of the building

The analysis made after fire showed numerous disagreements with the conditions contained in the standards that were adopted as the basis for the design of the existing sprinklers, especially considering requirements for the display of goods. Particularly improper height of storage, use of full shelves containing parts made of combustible materials, storage of cables and electrical wires without sprinkler protection, multilevel shelving, as well as too small distance between the shelves. Although goods intended for sale were laid, only to the height of 2.4 m, but the space above, to the height of up to 3.5 m, was used to store supplies of these goods, stored generally using combustible, paper packaging.

A separate issue, which also drew attention during the analysis, was numerous marketing campaigns carried out by the shop and intensified in the pre-holiday periods, to improve the economic result. The events led to installation of a variety of additional lighting, organized demonstrations, etc. The safety considerations in such situations often descended into the background, sometimes with many potential risks completely ignored.

The issue of paramount importance is to maintain the fire protection system capable of full readiness. The reports developed by fire departments for large retail stores, show that considered object was no exception. In many cases after installation the technical fire protection systems functioned correctly but after two or three years of operation it was turned out based on the individual opinions of the managers and was simply forgotten. Common practice is incomplete design documentation, missing operating manuals, maintenance reports, etc. This is ultimately the realty for many managers who in many cases run facilities with a collection of random documents, even mutually exclusive. In general there is a luck of information on the required maintenance practice. There was even a reported case of the modern large space trade building where the poorly maintained fire protection system at all did not work and the building burned completely. In the economical battle field, the owners and managers of such facilities generally are aware of their incompetence just in case the object of control by firefighters or representatives of insurance companies.

### 11.6.3 Lack of professionalism of technical staff

The described fire occurred after business hours. The facility was not occupied by customers or vendors. There were present only the security and maintenance staff. Must arouse astonishment, however, that there was no rescue action in the early stages of the fire. Particularly surprising is turning off several times the alarming devices activated at the appearance of fire. This behavior can be explained only by an illusory belief in the wrong message coming from the warning system. Undoubtedly, the staff was not prepared to undertake professional activities. Specialized safety training was carried out very rarely and was not effective. Besides, with the high turnover of the staff in this type of facilities, many workers by nature are subject to only to a typical amount of training at the adoption to work.

## 11.7 POTENTIAL EVACUATION FROM THE BURNING FACILITY.

The considered fire took place at night when the facility was without customers. However, on 14 fires that have occurred recently in this type of objects, 9 were in the daytime, occupied by the customers who need to be evacuated. In large sale-halls such evacuation involves large groups of people, usually composed of several hundred and often several thousand people. This can cause uncontrolled reaction and thus carries imminent risk of panic. It is not important in this case whether the fire became part of the roof insulation, shelving in the warehouse or exhibit at the show. Customers watching smoke spreading inside the sales floor or passage, and maybe even a fire, have the right to be concerned. On the one hand they are mindful of the sometimes overly dramatic media coverage of the other fires, and on the other hand without seeing any effective action against fire not only from employees but also be aware of not to the end efficient fire protection systems.

## 11.8 FINAL REMARKS

The statistics on major fires in large commercial buildings have been gathered in Poland for 36 years. It is very curious that for the first 32 years of observation there was only 4 such fires, while since 2008 already more than a dozen. This worrying trend is clearly related to the significant increase in this type of objects used in the country. Its explanation is not quite so simple. Certainly, the first raises the idea about the diminished alert. In practice, until 2008, the unwavering long-term operation of shopping centers has strengthened all: from investors to the owners and designers, and even the State Fire Service firefighters, in the belief that they found the right way to ensure a high level of safety of people and property at these facilities. Yes, from time to time, you could hear reports of fires in hypermarkets (for example Biuletyn, 1968), but always they took place quite far from Poland, and usually from Europe too. Yet surprisingly similar events began to appear also in our country, first sporadically, but then their frequency increased markedly. This increase was so pronounced that it was necessary to look for systemic causes, even though they do not always directly generate a fire hazard. It seems that the base problem is a change of investment strategy and practice. It is obvious that the primary criterion in this field is the project cost account. However, it should always be complemented by a credible risk assessment, taking into consideration the safety of people first. The more comprehensive shopping facility - service, the greater degree of difficulty in determining the method of its fire protection. So, it should not be uncritically accepted that the winners are the project teams that offer the cheapest service. The basis for selection should be experience in the design of such facilities and achieved results. Construction projects for hypermarkets should not be treated as completely reproducible projects. Despite many common features of their approach to the way of fire protection they should always be individualized. Must take into account not only the investor's objectives, but also local conditions, determining even the possibility of taking effective firefighting action. A reference list of designed and built objects should play a great importance in this case. A reasonable assessment of the protection system can be designed by independent auditors specialized in the provision of such services.

Very much prudence and discernment requires the use of appropriate solutions for the modernization of in-use facilities - those that will not only properly secure a new part of the building, but also unify, within reasonable limits, the level of fire protection in the whole store. One should not lead to a situation where not only in one fire zone, or in one passage but even in the sale room, there will be different operating conditions, valid for the same usage, such as different height of storage of goods. When designing an object one cannot forget about the next user who will not be required to have an expertise in fire protection and the resulting operating conditions. Hence there is a conclusion that the solutions should be transparent functional and friendly, not only for the owner and staff, but also for the firefighters themselves. One should also take into account the possibility of errors committed during the operation of the building and limited practical knowledge of those responsible for maintaining the facility and its protection system in good condition. On the other hand, facility personnel should be prepared so as to be able to execute any command related to the firefighting action, the technical infrastructure, yet provide a detailed explanation in this regard. This implies the need for periodic training, both in theory and in the form of practical exercises, designed to provide knowledge about the functioning of fire safety systems in the facility. To be effective in such training it is important to limit the scope with the knowledge to only that is needed for their position and required to properly perform their duties. Entrusting this type of training to carry out for random entities or individuals is pointless. The trainer should have full knowledge about the object and its fire safety system.

The last issue is how to make any changes in the facility, especially in the arrangement of its equipment. In shopping centers such situations occur, in principle, on an ongoing basis and you need to factor them in the safe operation of the system. Particular attention should be paid to the changes in land surface of the sales rooms and passages, how to display and store goods, including the type of shelving, changes in the communication system, the range of goods sold, etc. An important role should be assigned to monitor all kinds of marketing campaigns linked to the organization of temporary stands. The basis should be on the field to submit a project manager building the planned activities. This document should always be evaluated by a person competent and adequately prepared for that task.

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# 12 PERFORMANCE BASED DESIGN OF CENTRUM GALERIE DRESDEN

Summary

In this case study for a large shopping center it is demonstrated how escape routes can be optimized by a performance-based design with simulation of smoke propagation and evacuation analysis based on the fire safety concept of hhpberlin. The design for a superstructure of a road between two parts of the shopping center building was established with the natural fire method. In this way it is shown unprotected steel is feasible for the superstructure.

## 12.1 SCOPE

Large shopping centers imply a special challenge for the strategy of fire safety. Due to the manifold use (shopping, assembly hall, office, garage etc.), the individual demands of the users and the expansion of the building as well, fire safety concepts cannot be created only on a prescriptive way, for there are many deviations from the requirements of the building codes.

In fact such buildings require a performance-based fire safety concept. In this way an adequate safety level is to be proved by engineering methods.

In this case study for a large shopping center it is demonstrated how escape routes can be optimized by a performance-based design with simulation of smoke propagation and evacuation analysis based on the fire safety concept of hhpberlin. The design for a superstructure of a road between two parts of the shopping center building was established with the natural fire method. In this way it is shown unprotected steel is feasible for the superstructure.

The Centrum-Galerie Dresden dimensions are:

- max. length (direction East-West) 180 m
- max. width (direction North-South) 125 m
- max. height of the floor of a habitable room 20 m

Besides the actual shopping areas, there are planned garages, catering, offices, storage areas, plant areas and a center management.

## 12.2 PERFORMANCE BASED DESIGN FOR THE ESCAPE ROUTES

In the course of planning it emerged that the prescriptive substantial requirements concerning the widths of the rescue routes and the direction of the emergency routes were no realizable with the design for the “Centrum Galerie” in Dresden, Germany. To show this two examples are named:

- For shops with less than 100 m² the German Building Regulations for retail uses allows to lead both emergency escape routes over the mall. Shops with more than 100 m² sales area require a second escape route independent from the mall. On the 1st floor, the mall was shaped widely. Within the mall, covered shops were ordered which sometimes have a sales are of more than 140 $m^2$.
- In accordance with the German Building Regulations for retail uses for exits to the outside and stairwells designed for shops with more than 500 m² net area a width of at least 0.30 m is required in relation to each 100 m² of sales area.

The implementation of these requirements would have meant a number of additional staircases in the considered building and the conception of the central Mall would not have been realizable.

To meet the safety target requirements of "facilitating the rescue of people", a performance-based approach was chosen by way of deviance from the prescriptive design.

### 12.2.1 Simulation of smoke propagation

Using a simulation of the smoke propagation with the CFD model FDS (McGrattan, 2009) based on fire scenarios agreed upon with the approval authority, it has been demonstrated that the target criteria with regard to a low-smoke layer formulated in the German vfdb-guideline can be met for a sufficiently long period. Due to this document, the routing of both emergency escape routes via the mall could be approved of for shops smaller than 100 m².

The air space of the mall, for which the simulation of the smoke propagation was carried out, extends from the basement to the 4th floor, which is flush with the roof of the mall.

In the basement there is a ventilation opening for supply air of a total 5 m², the openings are arranged above the corridor doors and are automatically controlled by the fire detection system in the case of fire. On the ground floor, three arms of the main Mall branch off. On the ground floor, more than 40 m² of supply air ventilation openings are available, which are formed by the access doors. The branches are not separated from the main mall by smoke protection curtains. On the 1st floor, an amount of air of 15 $m^3/(h * m^2)$ is blown from the shops into the Mall as mechanically driven supply air. The mall ends in the 1st floor. The air space of the 2nd and 3rd floor of the mall is separated from the garage. A total of 12 exhaust fans arranged in the longitudinal walls of the roof of the mall on the 4th floor exhaust a total air flow of 350,000 $m^3/h$.

The results of the decisive fire scenario in the basement of the mall are shown below. Because the mall of the ground floor partly covers the mall in the basement, the fire in the simulation was placed so that the fire plume come up against the ceiling above thus covering the worst case scenario. The heat release rate was determined due to the $t^2$-approach with $t_g$ = 300 s. A constant lapse of the heat release rate was assumed after the sprinkler had been activated (see Fig. 12.1).

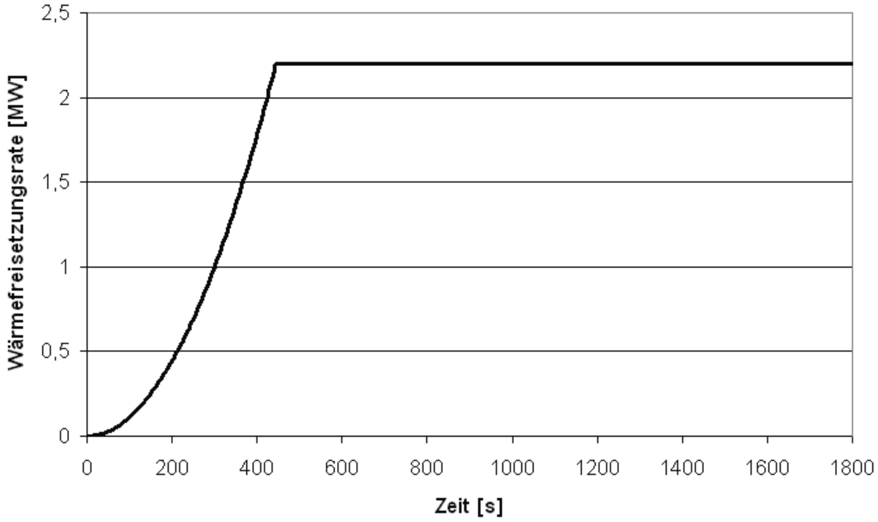


Fig. 12.1 Heat Release Rate for simulation of smoke propagation

The values of the visibility range after 1800 s on the first floor at a height of 2.50 m are shown below as an example.

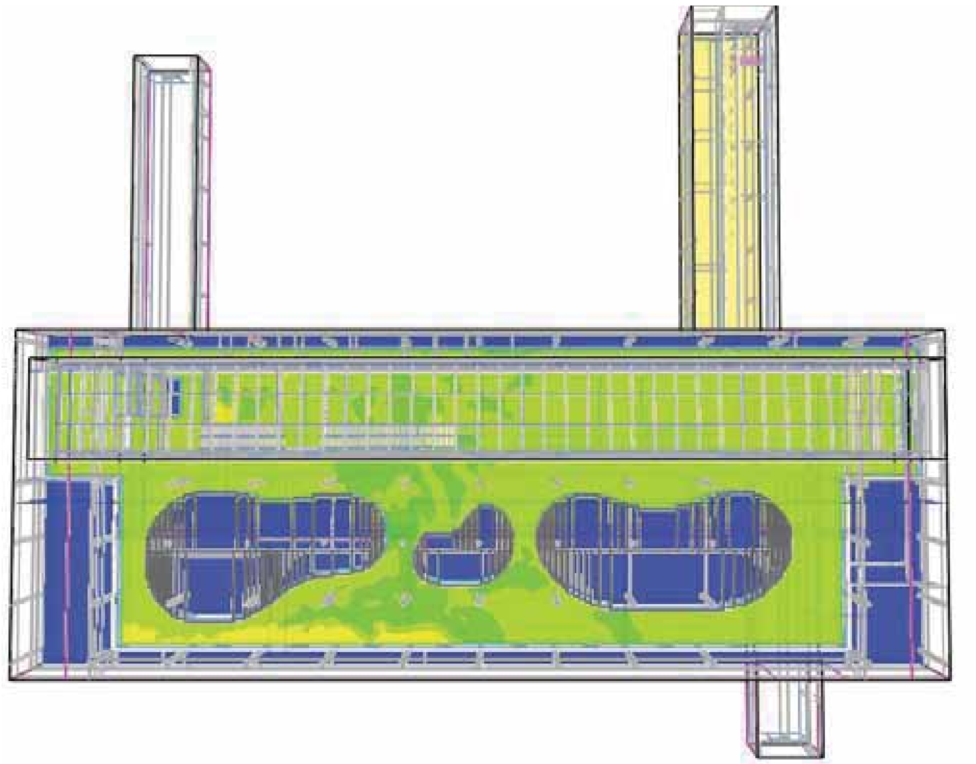

Fig. 12.2 Visibility on the first floor at H = 2.50 m after 1800 s

In a stationary condition visibility ranges on the first floor are still between 10 m to 15 m after 1800 s. Only in locally limited areas the visibility is slightly below 10 m.

### 12.2.2 Evacuation Analysis

The occupancy was determined in accordance with German vfdb guidelines (vfdb-Leitfaden TB 04/01) as follows:

- Ground floor 0.5 persons/m²,
- Remaining floors 0.3 persons/m²,
- Office areas 0.2 persons/m².

This results in an overall occupancy of 14,510 people for the evacuation scenario of the entire shopping centre (excluding the garage).

The reaction time is calculated to be 2-5 minutes following Purser (vfdb-Leitfaden TB 04/01).

The proof of a sufficient capacity of the emergency exits was led with an evacuation analysis (buildingEXODUS V4.06, 2006).

In the first minutes of the evacuation, short-term congestion may arise in the area of the exits (see Fig. 12.3).

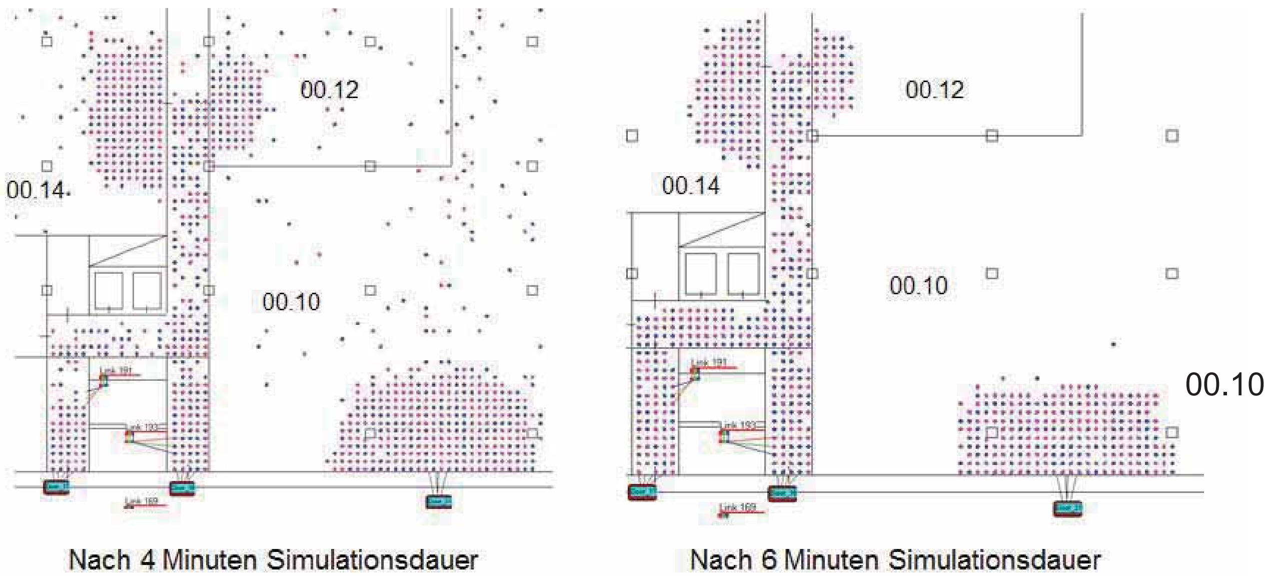


Fig. 12.3 Short-term congestion at ground floor exits

A total of 10 simulations were carried out. On average there were the following results:

- The whole building evacuated after 17 minutes,
- Most of the shops evacuated after 14 minutes,
- Last floor evacuated after 15 minutes.

The safety objective to avoid waiting times in the shops longer than 3 minutes, agreed upon with the approval authority, is achieved. Congestions on escape and rescue routes from the entrance onwards to the staircases only occur to a limited extent. A tailback into the flight of stairs does not happen.

The evacuation is completed 17 minutes after the start of the fire. The simulation of the smoke propagation has shown that for a sufficient period of time (> 20 minutes) the safety objective criterias according to (vfdb-Leitfaden TB 04/01) are fullfilled so that the performance based verification of the sufficient capacity of the rescue routes and the routing of both emergency escape routes from shops > 100 m² via the mall could be supplied.

### 12.3 PERFORMANCE BASED DESIGN OF THE SUPER-STRUCTURE OF THE TROMPETERGASSE

The Trompetergasse is located between part 1 and part 2 of the Centrum Galerie in Dresden. The Trompetergasse should be roofed over to allow customers and visitors to stroll between the two buildings also in bad weather.

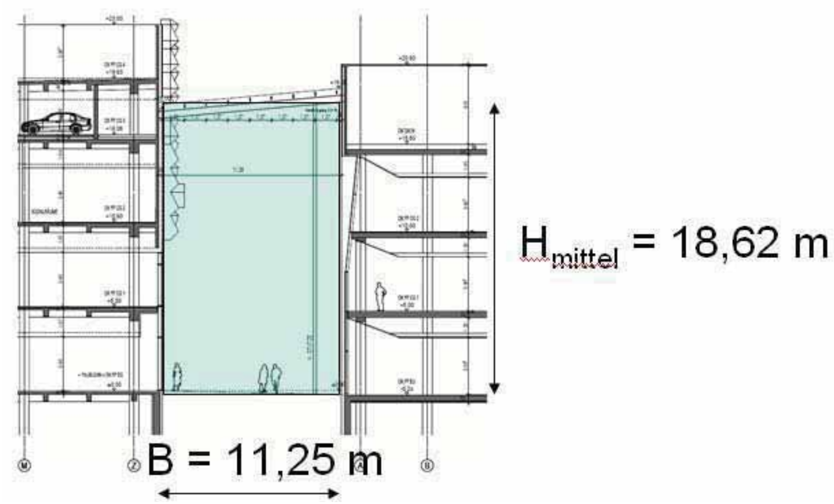


Fig. 12.4 The superstructure of the Trompetergasse

The roof is supported by the outer walls of the two adjacent buildings as well as a steel column. The roof itself consists of a steel structure with glass elements.

The area under the roofing is to be treated similar to the open. Therefore it remains to be proven that it is safe when people flee from the adjacent buildings into this street or when windows of staircases open in the direction of the Trompetergasse.

Generally, the area below the roof is only intended as a public thoroughfare. But because it cannot be ruled out that this area might be used with vehicles occasionally or that it is temporarily used in the course of specific events and the Trompetergasse will be used as access for the fire brigade, it should be verified by a performance based design that in case of a fire neither smoke will fill the Trompetergasse nor will the stability of the supporting steel structure fail.

In agreement with the approval authority a design fire with 15 MW rate of heat release was taken as the relevant fire scenario.

The following ventilation openings are present:

- At the junction of the roof and part 2 of the building a non-closed gap of 20 cm was taken into account
- Both facing surfaces of the covered area were assumed to be open across the full width.

The structural fire safety design of the superstructure was carried out by the simplified natural fire model following Eurocode 1 part 1-2 annex C (Heskestad plume) resp. the German vfdb guideline (vfdb-Leitfaden TB 04/01).

For the design of the superstructure the following assumptions were assumed:

- height of the fire above the ground: 2.50 m,
- minimum height of the superstructure above the ground: 17.40 m,
- maximum rate of heat release: $\dot{Q}_{\max}$ = 15 MW.

The temperature of the plume is calculated according to (vfdb-Leitfaden TB 04/01) as follows:

$$T_P = T_\infty + 25{,}5\frac{\left((1-\chi_r)*\dot{Q}\right)^{2/3}}{z^{5/3}} \qquad (1)$$

with: $T_\infty$: ambient temperature [K],

$\chi_r$: radiative fraction of the heat release rate [-],

$\dot{Q}$: rate of heat release [kW],

$z$: height [m].

Using a rate of heat release of 15 MW, a radiative fraction of 30% and a height z of 17.40 m - 2.50 m = 14.90 m, the plume centerline temperature can be calculated to 155 °C

Because the plume mass flow does not form unobstructed and contrary to the basic assumption hot gas instead of cold gas is mixed into the layer of hot gas after entering the mass flow, in (vfdb-Leitfaden TB 04/01) a suitable method is presented to determine the corrected temperature of the hot gas. The calculation is done by adding a plume temperature difference to the temperature of the layer of hot gas.

After the calculation a temperature difference of $\Delta T_{Plume}$ = 50.3 K has to be added to the hot gas temperature. The maximum hot gas temperature of 140 °C was calculated using the multi room zone model CFAST. Thus the corrected temperature of the plume is calculated:

$T_{Plume}$ = 140 + 50.3 = 190.3 °C.

The corrected temperature of the plume of 190.3 °C is higher than the uncorrected temperature of the plume and is therefore decisive for the calculation.

The determined temperature of the plume of 190.3 °C lies significantly below the critical steel temperature of approx. 500 °C. Thus, no special measures are required to protect the steel sub-structure of the superstructure.

Acknowledgements

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# 13 FIRE SAFETY DESING WITHIN RETAIL - BUYING 'MORE' FOR 'LESS' BY MAKING INFORMED DECISIONS

Summary

One of the most crucial aspects of fire safety design, is ensuring that this can be effectively implemented and managed during the buildings occupation. Having expensive and complex systems may solve a particular design issue, but ongoing maintenance and management involvement may result in this system being worthless or misunderstood by the buildings operators. This case study outlines one particular example where fire engineering has played a crucial role in improving the overall safety of an existing supermarket. In utilising a straight, forward-thinking approach an argument was created to remove an unwanted, existing and problematic natural smoke ventilation system in the roof where this did not work as intended. This system didn’t provide any fire safety benefit, but caused continued problems with water tightness and heat loss.

## 13.1 INTRODUCTION

This case study focuses on one particular project where an existing supermarket was being modernised, and extended. The overall floor plate was being increased from around 7,700m² to 8,100m² with a new mezzanine floor containing a coffee bar, and back-of-house areas. One of the main aspects of this building was the existing system of manual smoke ventilation located within the ceiling space. This was installed within the original building some 10 to 15 years previously due to a local county building code. The code requested that to facilitate firefighting operations, a certain percentage of the floor space should be provided with natural ventilation for smoke clearance. These had been successfully installed within the roof space, but due to the lack of staff training and understanding of how the system should operate these were left to slowly fall into a state of disrepair.

This concern was addressed within the newly written fire safety strategy, and a new qualitative approach was adopted which took into consideration management capabilities, onsite fire safety equipment, the internal height of the store and Fire Service access provisions to, and around the building. UK’s most recent fire safety design code, BS9999, was applied throughout the building. This code is written in a such a way, that it allows the extension of travel distance, optimisation of stair and exit widths when taking into consideration additional fire safety features which may be present within the building such as:

high ceiling heights which would act as a smoke reservoir; a higher standard of automatic alarm and detection; management levels which may be in place.

## 13.2 FIRE STRATEGY OUTLINE

The main purpose of the fire strategy was to create an 'easy-to-use' end product, which could be implemented into the day-to-day running of the supermarket. By adopting this, there would be a greater degree of certainty that the fire strategy would work as intended when required.

### 13.2.1 Legislative Requirements

A number of guidance documents had to be consulted during the fire safety design process. The main guidance document which was used to design the fire strategy was BS 9999: 2008. This document was the preferred choice, as it takes into consideration a risk profile made up of the fire growth rate and occupancy type associated with the building. This risk profile was then used to determine items such as travel distances and exit widths. Travel distances were extended by almost 30% further than the previously applied method which used an older standard, Approved Document part B.

Due to the location and size (containing a volume greater than 7,000m$^3$) of this supermarket, a local building act also applied. The primary reason for this local act is to reduce the risk of fire spread within large 'warehouse' type buildings, and to increase provisions for Fire Service operations. It was this legislation which originally called for the provision of natural roof ventilators within the roof space.

Lastly the supermarket's own fire safety requirements had to be considered. This set of minimum design standards had to be incorporated, including items such as: minimum fire alarm classifications; automatic suppression, and the incorporated of compartmentation to specified areas.

### 13.2.2 Means of Escape

A full means of escape assessment was carried out on the proposed building, with the aim to make provisions more efficient and easier to manage. The existing supermarket relied heavily on occupants escaping through back-of-house areas to meet the original 45m travel distance limit from front-of-house areas. This is not a desired method of design as it relies heavily on each staff members' ability, ensuring that goods are not stored within dedicated escape routes - at all times. In addition to this, it is known that occupants have a higher degree of reluctance to escape through back-of-house areas where they are normally forbidden to enter – adding to the risk associated with escaping through staff only areas.

During a site visit and a meeting with the supermarkets management, it was clear that maintaining clear routes through back-of-house areas was difficult to implement due to continual stock rotations, lack of staff responsibility and general shortage of storage space.

Fig. 13.1 Back-of-house

It was one of the main aims to remove these escape routes from the new design, and an assessment was carried out in accordance with BS 9999: 2008 to study any alternatives. Taking into consideration: ceiling heights; a high level of alarm category (incorporating a spoken instruction system); an automatic sprinkler system being upgraded throughout the building, travel distances were increased to a maximum of 60m where two means of escape were available. This is a vast improvement on the original design, as the means of escape routes could now be diverted away from the back-of-house areas where existing final exit doors were wide enough for the proposed occupant numbers.

In Fig. 13.2, travel distances through back-of-house areas are indicated in blue, where these are restricted to a maximum of 45m when designing to Approved Document, part B. The red lines indicate where travel distances have now been designed to a maximum of 60m enabling means of escape to be possible without travelling through staff only (back-of-house) areas.

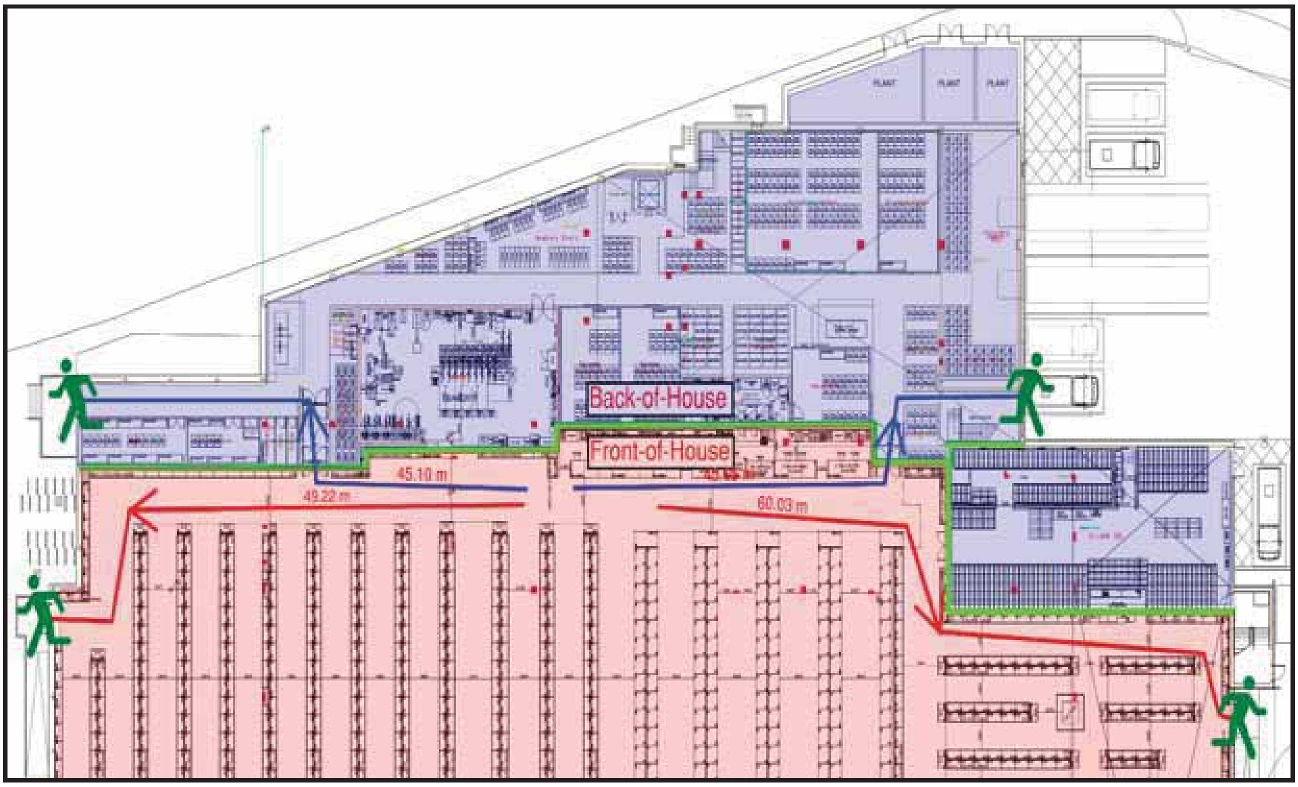


Fig. 13.2 Escape routes diverted away from BOH areas

### 13.2.3 Fire Service Access

Fire Service access to (and around) the building is a crucial factor, as the building should be easily accessible allowing the fire to be fought in an efficient and realistic manner. One of the main benefits of this supermarket was its location relative to surrounding access roads. These roads provide just over 90% perimeter access around the buildings' circumference enabling the local Fire Service to gain access around, and into each elevation via suitably positioned access doors.

Fig. 13.3 Perimeter access - image sourced from Google Maps

Fig. 13.4 Areal view of supermarket including perimeter access (red) - image sourced from Google Maps

**13.3 REMOVAL OF SMOKE CLEARANCE VENTILATION**

The existing smoke ventilation system located on the roof was primarily intended for use by the Fire Service to clear smoke during firefighting operations. This system of natural smoke vents caused substantial problems for the supermarket, and was never fully understood. Over time, this system began to leak allowing rain water to fall through spoiling products below at a considerable cost to the supermarket.

The supermarkets management made the decision to install a series of rain catching plastic sheets, and associated water drainage pipes to stop products from being spoiled during every rainfall. This decision ultimately deemed each of the vents as ineffective, as they had in some areas been fully blocked-up.

To further complicate matters, the store was in breach of the law by not maintain their fire safety equipment. In accordance with the Regulatory Reform (Fire Safety) Order 2005, the responsible person i.e. the supermarket manager was at serious risk of being prosecuted.

Fig. 13.5 Roof ventilation covers

Due to the failure of this system, it was seen as a necessary step to review the entire smoke ventilation strategy within the building. It was known that these ventilators were not required for life safety, however a further argument had to be developed providing evidence that Fire Service operations would not be compromised should these be removed.

By developing a qualitative argument that considered the overall on-site Fire Service provisions, consent was gained from the approving authority that these smoke vents were not required for firefighting purposes.

The building had an excellent degree of Fire Service vehicular access where a perimeter road was available to at least 90% of the buildings perimeter. This is compared to 50%, which is the minimum as recommended within BS9999:2008. The existing building was also provided with an automatic sprinkler system; in its original arrangement the sprinkler system was fed of the town main which is not current practice as this supply is not guaranteed to provide a minimum water flow and pressure. By upgrading the water supply to a full capacity tank to fully comply with BS EN 12845:2004 and extending this to all areas within the building which were previously not covered, this system was being brought up to a modern day standard. The presence of an automatic sprinkler system with this supermarket offers a major benefit in terms of controlling a fire in its early growth stages, and restricting the spread of this within the building. This sprinkler system would inherently present the Fire Service with a smaller fire size in comparison with an unsprinklered fire, where access to the building may not even be possible due to the severity of a fast fire growth rate associated with retail.

The layout of this supermarket shelving was also considered in relation to the overall building height. The standard shelving system has a height of around 1.8m with a distance between shelves of no less than 2.7m. This shelving configuration would offer a restriction on the possibility of fire spread. One of the major benefits of this supermarket renovation was the proposals to increase the floor-to-ceiling height throughout.

A false ceiling was being removed, increasing the floor-to-ceiling height from around 2.5m to 3.4m. In addition to this a new double height space was being created towards the front of the supermarket offering a new entrance space. This space was up to 8.2m in height offering a larger volumetric space for smoke to accumulate within.

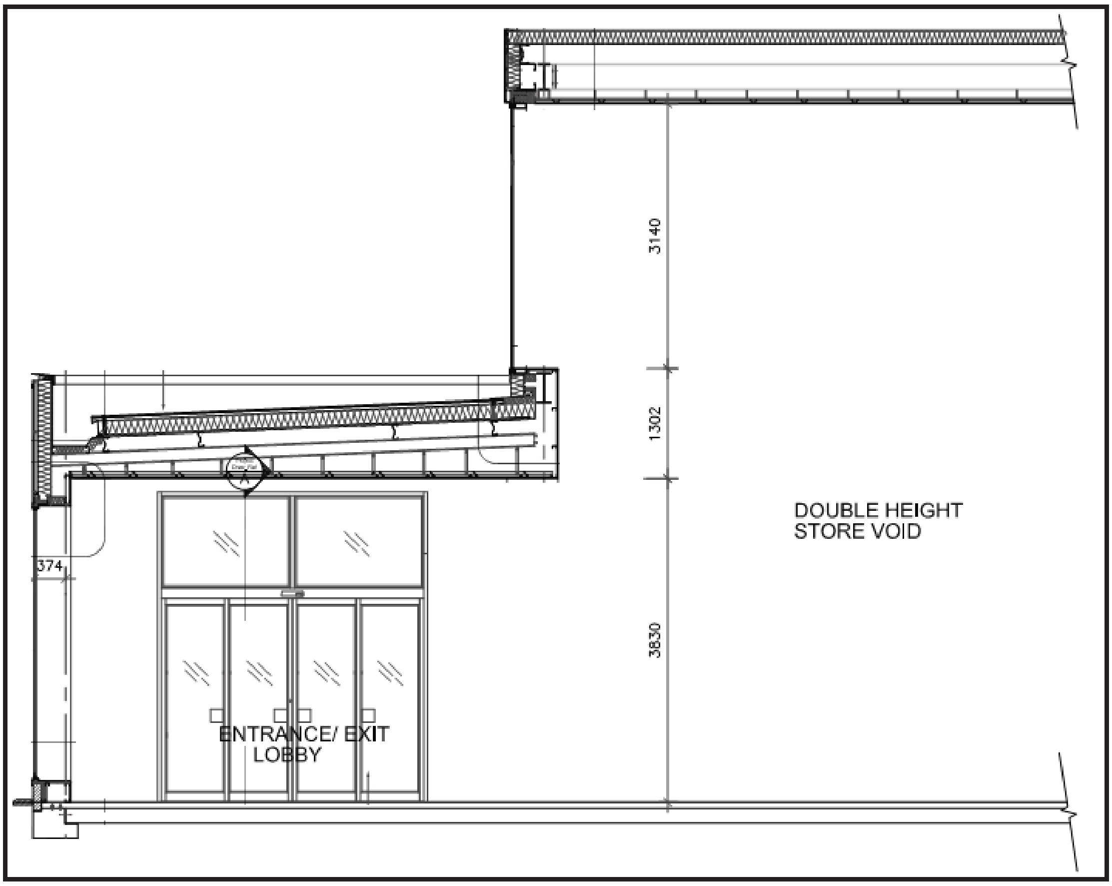


Fig. 13.6 New double height entrance space

### 13.4 CONCLUSION

By selecting a modern code that considers fire safety systems including: the internal floor-to-ceiling height; automatic fire alarm category and management levels, it was possible to redesign the means of escape provisions to make these more efficient and easier to manage on a day-to-day basis. In addition to this, it was also possible to design out the leaking natural smoke ventilation system by compiling a qualitative argument.

Maintaining clear exit routes and maintaining the smoke ventilation system proved very difficult to manage, and put the responsible person and the supermarket at risk of prosecution under the UK's Regulatory Reform (Fire Safety) Order which controls fire safety in occupied premises.

The means of escape assessment did not require complex numerical models and extensive parametric studies to address uncertainties within the design. The selected code which dealt with this (BS 9999: 2008) is sufficiently flexible, and offers clear benefits when taking into consideration aspects such as extensive ceiling heights which act as smoke reservoir space. As such, the uncertainty associated with deterministic modelling was avoided and a significant improvement implemented by removing complexity from the fire safety management scheme, at very reasonable cost to the client.

Acknowledgement

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# 14 EVALUATION OF THE FIRE RESISTANCE OF THE STEEL STRUCTURE OF AN EXHIBITION CENTRE USING STRUCTURAL FIRE SAFETY ENGINEERING

Summary

This case study reports on the needs of passive fire protection to ensure fire resistance requirements of the steel structure of an exhibition centre. Due to the large dimension of the exhibition centre, with an average height of 13 m and a surface of about 6500 m$^2$ a prescriptive approach using the standard fire curve ISO834 for a required fire resistance of R120 revealed to be too severe, unrealistic and uneconomical. The analysis was made using the advanced calculation models allowed by the fire parts of Eurocode 1 and Eurocode 3. A performance-based analysis shows, in this study that, protecting the structure for a standard fire resistance of 60 minutes (R60), considering a critical temperature of 500ºC, the load-bearing function is ensured during the complete duration of the natural the fire scenarios used, including the cooling phase. Using a prescriptive approach and without making any calculation, the steel structure should have been protected, according part 1-2 of Eurocode 3, for a critical temperature of 350ºC and for R120 (the required fire resistance according the occupancy and the dimension of the building).

## 14.1 INTRODUCTION

It is the purpose of this paper to present a study performed on the fire resistance of the steel structure of an exhibition centre.

In the Portuguese Technical Regulations for Buildings Fire Safety, on the Decree No. 1532/2008 (MAI, Regulamento Técnico, 2008 and MAI, Regime Jurídico, 2008), which is now implemented, two approaches are recommended for assessing the safety of structures exposed to fire: a prescriptive approach using the standard fire curve ISO 834; and a performance based design using the natural fire development concept. The natural fire curve definition takes into account the size of the fire compartment, the ventilation conditions and the thermal properties of the fire compartment lining materials, in opposition to the standard fire curve that does not depend on any of these parameters.

In addition, in the last years several European projects (European Commission, 1999a, European Commission, 1999b, European Commission, 2007) have shown that in large compartments, the prescriptive regulation based on the standard fire curve is too conservative and unrealistic.

According to Part 1-2 of Eurocode 3 (EC3), the stability verification can be made verifying that:

a) with the standard fire, the structure collapse does not occur before the fire resistance time defined by the regulation; or

b) with the natural fire and advanced calculation methods the structure collapse does not occur during the complete duration of the fire, including the decay phase or during a required period of time, which may coincide with the fire resistance time defined by the regulation.

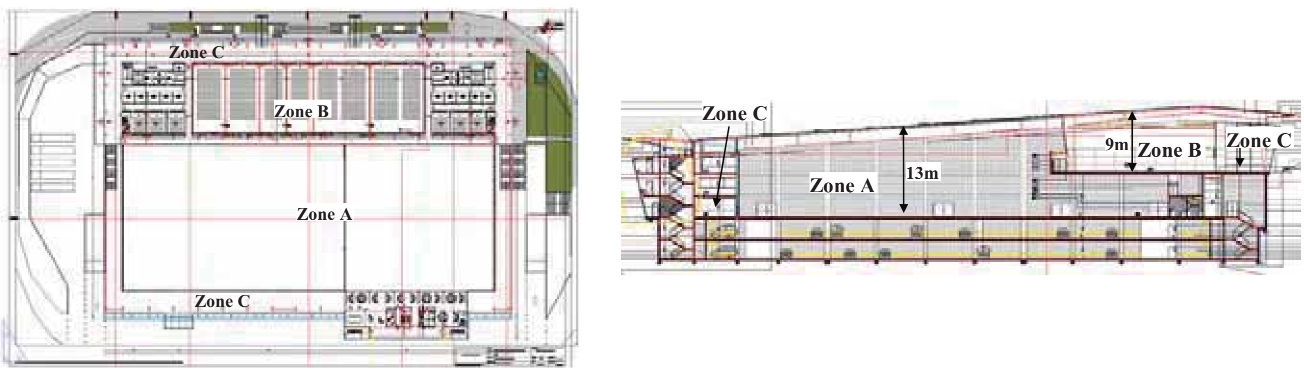
In this work, the studies, performed to assess the fire resistance of the steel structure of an exhibition centre in Oeiras (Portugal), are presented.

Advanced calculation methods were used (Franssen, 2010). For the natural fire simulation the programme Ozone (Cadorin, 2003a, Cadorin, 2003b), developed at the University of Liege was used and to simulate the thermo-mechanical behaviour the finite element program SAFIR (Franssen, 2005) also developed at the University of Liege has been used.

It was also considered the occurrence of possible localized fires, according with Part 1-2 of Eurocode 1 (EC1). This methodology from Eurocode was implemented in the program Elefir-EN (Vila Real, 2010) (developed at the Universities of Aveiro and Liege).

The fire compartment temperature definition was determined, as defined in Part 1-2 of EC1, with each of the following fire models: the localise fire and 1 or 2 zone models, according to whichever is more appropriate. These models correspond to different types of fire and different phases of the same fire.

According to the Portuguese fire regulation the Oeiras Exhibitions Centre (see Fig. 14.1), is considered to be of the Utilization-type VI « Theatres/cinemas and public meetings ». As its height is less than 28 m, has two floors below the reference plane and an effective is higher than 5000, it is classified with the 4th Risk Category. According to this regulation, the main elements of the structure must have a standard fire resistance of 120 minutes (R120). However, given their large plan dimensions and height, it can be classified as atypical hazardous being allowed the use of solutions of Fire Safety Engineering using performance- based design.




a) b)

Fig. 14.1 The Oeiras Exhibitions Centre a) plan; b) cross section

The Oeiras Exhibition Centre is, due its dimensions in plan and height, qualified of "Large compartment" (European Commission, 1999a). Moreover, the fact that the main structure is made of steel elements of class 4 cross-section (EN 1993-1-1), a prescriptive analysis would force the use of passive protection against fire designed for a critical temperature of 350°C and a fire resistance R120, as prescribed in EN 1993-1-2, if no structural analysis was performed.

This study, on the steel structure fire behaviour, aimed at determining the fire resistance of the building structure.

## 14.2 FIRE SCENARIOS

For definition of the most likely fire scenarios, it was considered three distinct zones shown in Fig. 14.1: the area for exhibitions and fairs (Zone A), area of auditoriums (Zone B) and the passage surrounding area (Zone C).

The temperature evolutions were determined using the calculation software Ozone V2.2 (Cadorin, 2003a, Cadorin, 2003b). The definition of the natural fire curves took into account the fire compartment dimensions, the ventilation conditions, including openings corresponded to the smoke exhaustion system (electrical control, considered calibrated to 70 °C) and walls lining materials, in opposition to standard fire curve that is independent of these parameters. Although, it was considered the beneficial effect of the use of sprinklers, other additional active fire fighting measures, such as the existence of fire detectors and alarms, automatic warning supported by the public phone network connected to the fire brigade, fire fighting devices among others, were not, on the safe side, took in to account when defining the fire load design value to be used.

### 14.2.1 Compartment fires

Although the EC1 allows the consideration of the beneficial effects of various measures of active fire safety, this study was chosen, as previous mentioned, by considering only the effect of sprinklers, and the design value of the fire load density value of is defined by

$$q_{f,d} = q_{f,k} \cdot m \cdot \delta_{n1} \qquad \left[MJ/m^2\right] \tag{1}$$

where: $q_{f,k}$ is the characteristic value of fire load density per unit area of the floor [$MJ/m^2$]; m is the combustion factor, considered in this study equal to 0.8; and $\delta_{n1}$ the factor that considers the effect of sprinklers, equal to 0.61.

The adopted fire load density characteristic values were:

- Rooms for the exhibitions (Zone A): $q_{f,k}$ = 400 $MJ/m^2$ (European Commission, 1999a)
- Auditorium (Zone B): $q_{f,k}$ = 365 $MJ/m^2$ (EN 1993-1-2)

- Fire located in the circulation zone (Zone C): $q_{f,k}$ = 1824 MJ/m$^2$ (same as for libraries), (EN 1993-1-2)

On the surrounding circulation zone (Zone C) it was admitted the possibility of occurrence of a localized fire in a stand for exhibition and books selling with an area of 12 m$^2$, which corresponds to the most severe situation in the fire load densities table given in EN 1991-1-2.

The openings (corresponding to the doors) were considered completely opened from the beginning of the fire (most severe situation resulted from a parametric study considering various openings percentages). It was also taken in to account in Zone A the existence of 21 smoke evacuation systems, with a net surface of 1.96 m$^2$, electrical controls and calibrated to 70 °C, being 12 located in the largest hall and 9 on the smaller. It was assumed that the walls were composed of concrete blocks. The cover was built of sandwich panels with profiled steel sheeting of 0.75 mm thick with rock wool of 40 mm thick. It was also used a rate of heat release $RHR_f$ = 500 kW/m$^2$ (EN 1993-1-2) with a fast fire growth rate $t_\alpha = 150$ s (EN 1993-1-2).

### 14.2.2 Fire scenarios definition

The considered compartment fire scenarios are presented in the following sections.

Zone A: exhibition halls

The Zone A (see Fig. 14.1) is divided into a large hall and a small hall. Thus three different fire scenarios were considered: fire in the large hall, fire in the small hall, and fire in two halls simultaneously. In these scenarios the average height is of 13.0 m, and the considered fire areas are present below.

**Scenario 1** - Fire in the larger hall
The considered maximum area was $A_{f,max}$ = 6525 m$^2$, and the fire area $A_{fi}$= 6525 m$^2$.

**Scenario 2** - Fire in the smaller hall
The considered maximum area was $A_{f,max}$ = 4410 m$^2$, and the fire area $A_{fi}$ = 4410 m$^2$.

**Scenario 3** - Fire in the two halls simultaneously
The considered maximum area was $A_{f,max}$ = 10935 m$^2$, and the fire area $A_{fi}$ = 10935 m$^2$.

The temperature evolutions of these compartment fire scenarios are plotted in Fig. 14.2a. The most severe is scenario 2.

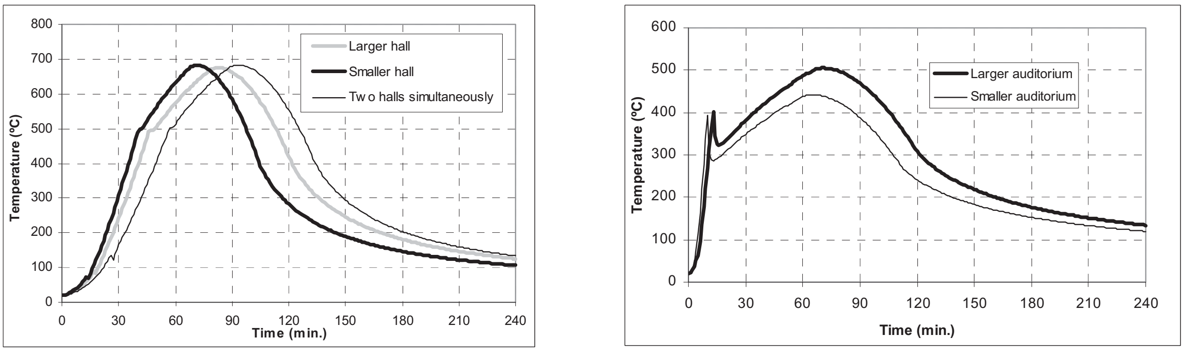
Zone B: Auditoriums

Zone B (see Fig. 14.1) is composed of several auditoriums. Two different fire scenarios were considered: fire in the smaller auditorium and fire in the larger auditorium. In these scenarios the average height is of 9.0 m, and the considered fire areas are present below.

**Scenario 4** - Fire in the smaller auditorium
The considered maximum area was $A_{f,max}$ = 194.88 m$^2$, and the fire area $A_{fi}$ = 194.88 m$^2$.

**Scenario 5** - Fire in the larger auditorium
The considered maximum area was $A_{f,max}$ = 409.92 m$^2$, and the fire area $A_{fi}$ = 409.92 m$^2$.

The temperature evolutions of these compartment fire scenarios are plotted in Fig. 14.2b. The most severe is scenario 5.




a) b)

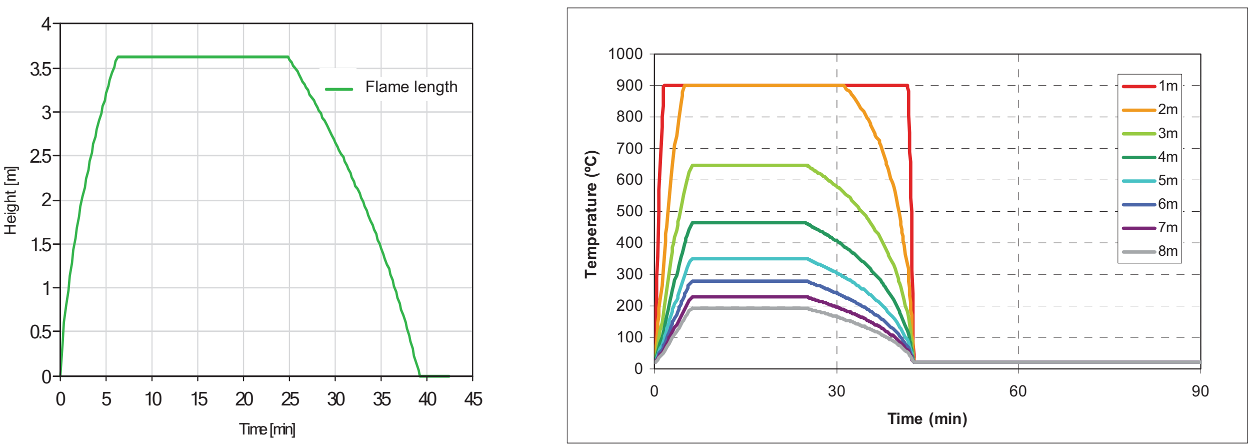
Fig. 14.2 Temperature evolution in the compartments: a) Zone A; b) Zone B

Zone C: Localized fire in the surrounding circulation zone

Zone C (see Fig. 14.1) is the circulation zone. As previously mentioned, it was considered the possibility of a localized fire, resulting in only one fire scenario. It was calculated the maximum height of the flames, in case of a localized fire according to the EC1 part 1.2 based on the Heskestad model (Heskestad, 1983). In this scenario the minimum compartment height is of 9.0 m and the maximum height of 11.4 m, the considered fire area is present below.

**Scenario 6** - Localized Fire in the circulation zone
The considered maximum area was $A_{f,max}$ = 12 m$^2$, and the fire area $A_{fi}$ = 12 m$^2$.

Fig. 14.3 shows the evolution of the flame length and the temperature development for different heights, obtained with the program Elefir-EN (Vila Real, 2010).




Maximum length of the flames: 3.63 m

a) b)

Fig. 14.3 Scenario 6: a) flame length; b) temperature development at different heights

### 14.3 MECHANICAL ANALYSIS

A 3D mechanical analysis using the software SAFIR (Cadorin, 2003b), with shell finite elements was used.

In the structural analysis the portal frame shown in Fig. 14.4 was used, which was representative of all the main frames. The frame comprises two main spans, one span with a length of 60 m (beams V2, V3 and V4), and another with 30 m length (beams V5 and profile IPE270).

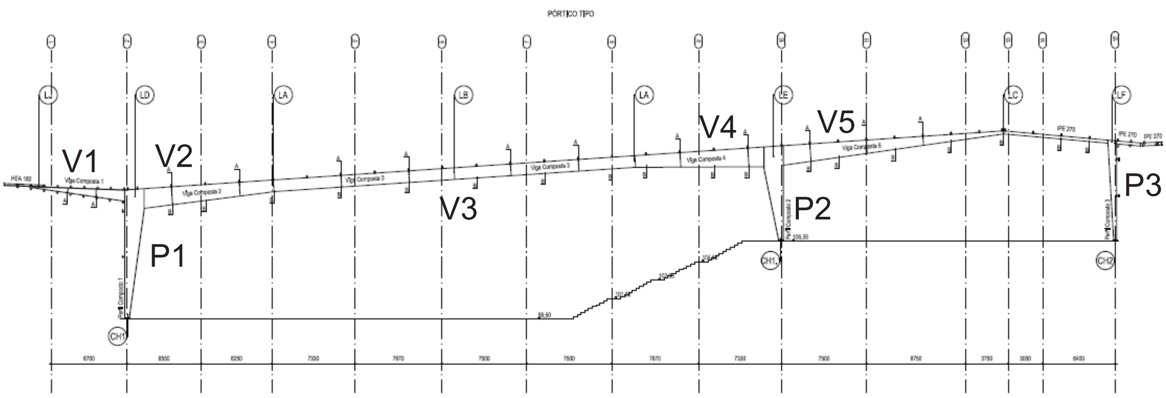



Fig. 14.4 Analysed portal frame

#### 14.3.1 Mechanical actions

Fire is considered an accidental action, which means that the design value of the action effects in fire situation, should be obtained using an accidental combination as defined in EN 1990 and according to the Portuguese National Annex of the EN 1991-1-2:

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

where $G_k$ refers to the permanent loading and $Q_k$ to the variable action.

The roof loads were determined, in accordance with Annex A1 of the EN 1990, adopting the category H for roofs, which corresponds to $\psi_1 = 1.0$ and $\psi_2 = 0.0$ for the wind actions and $\psi_1 = 0.0$ and $\psi_2 = 0.0$ for roof imposed loads.

For the building roof and facades 3 load combinations were adopted: Combination 1 - $1.0G_k$; Combination 2 - $1.0G_k + 0.2W_1$; and Combination 3 - $1.0G_k + 0.2W_2$, where $W_1$ and $W_2$ are the wind actions in the two orthogonal directions.

#### 14.3.2 Cross-sections thermal analysis

The thermal analyses of the cross sections were performed with the program SAFIR. From these analyses it was obtained the temperature field of the cross sections for each of the considered fire scenarios, which was later applied to the mechanical analysis.

The analysed structure is composed of class 4 I-sections (EN 1993-1-1). For example Fig. 14.5 shows the cross sections of two beams, one of non-uniform section (see V2 in Fig. 14.4) and another with uniform section (see V3 in Fig. 14.4). The fact that the sections are of non-uniform and of class 4 justified the use of structural modelling with finite shell elements, as illustrated in Fig. 14.5. The thermal analysis was, thus, performed with one-dimensional element for each different thickness and not for the entire cross-section. The used thicknesses varied between 6 mm and 18 mm in the profiles and reached 55 mm in the connections end plates.

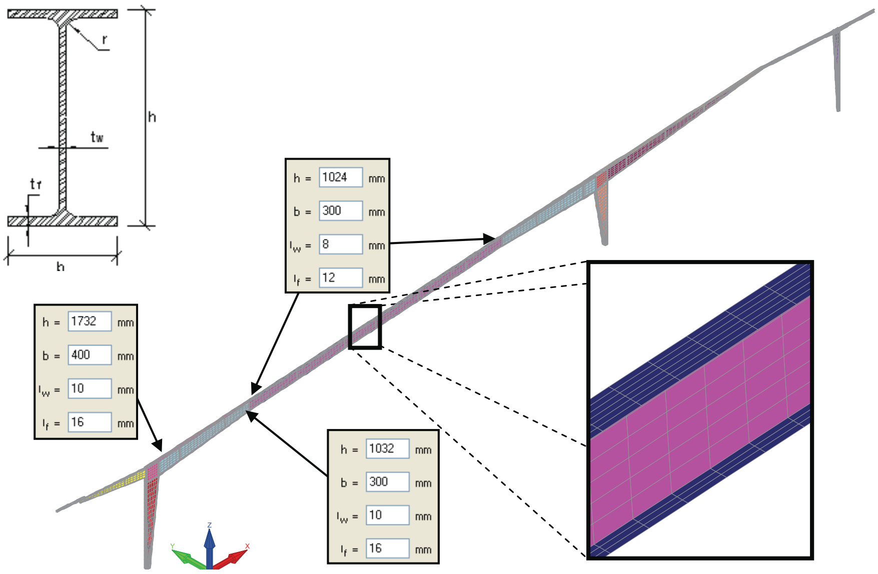


Fig. 14.5 Examples of the analysed cross-sections

It was considered, in the safe side, that all the shell elements were subjected to fire on their two sides, corresponding to fire on the four sides of the I cross-sections.

During the mechanical analysis it was found that it was necessary to protect the cross-sections to fire. The insulation thicknesses were chosen so that they would ensure fire resistance to the standard ISO834 curve for 60 minutes, considering a critical temperature of 500°C. Fig. 14.6 shows the temperature evolution in the steel sections with and without protection, due to fire scenario 1.

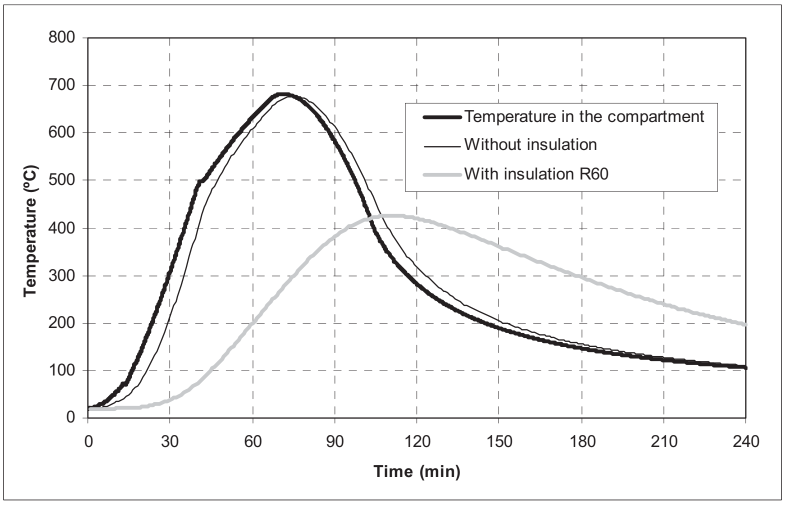


Fig. 14.6 Steel temperature evolution with and without protection

### 14.3.3 Analysed structural system

The structure was made of the steel grade S355. As mentioned above the structure has been modelled with shell elements. Fig. 14.7 shows the used model with some beam to column connections details.

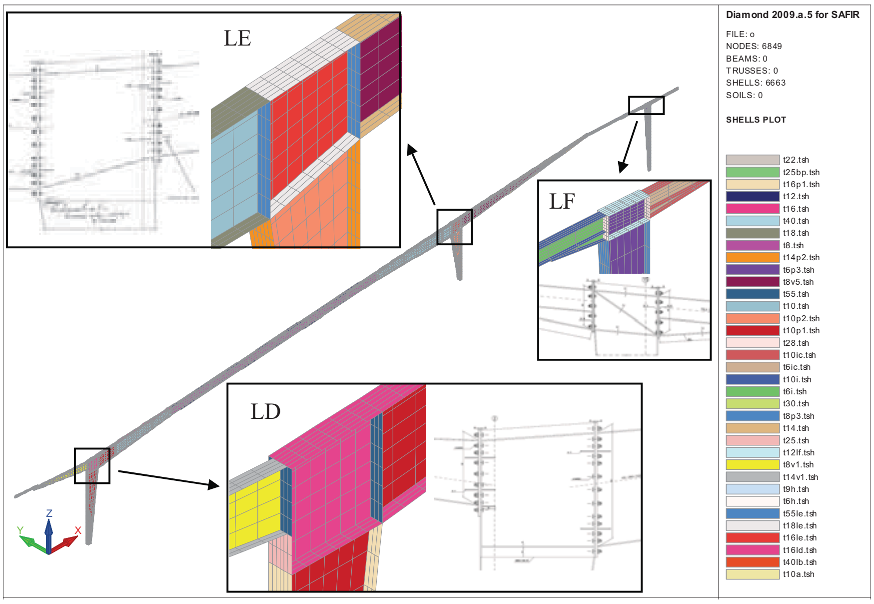



Fig. 14.7 Structural model and beam to column connection details

Fig. 14.8 shows the introduced restrictions to the structural model. It was considered that on the columns bases, the supports prevented the translations in all directions. Restrictions perpendicular to the frame have also been adopted. Figure 8 also shows the details of the beam to beam connections and of one of the transverse stiffeners considered in the analysis.

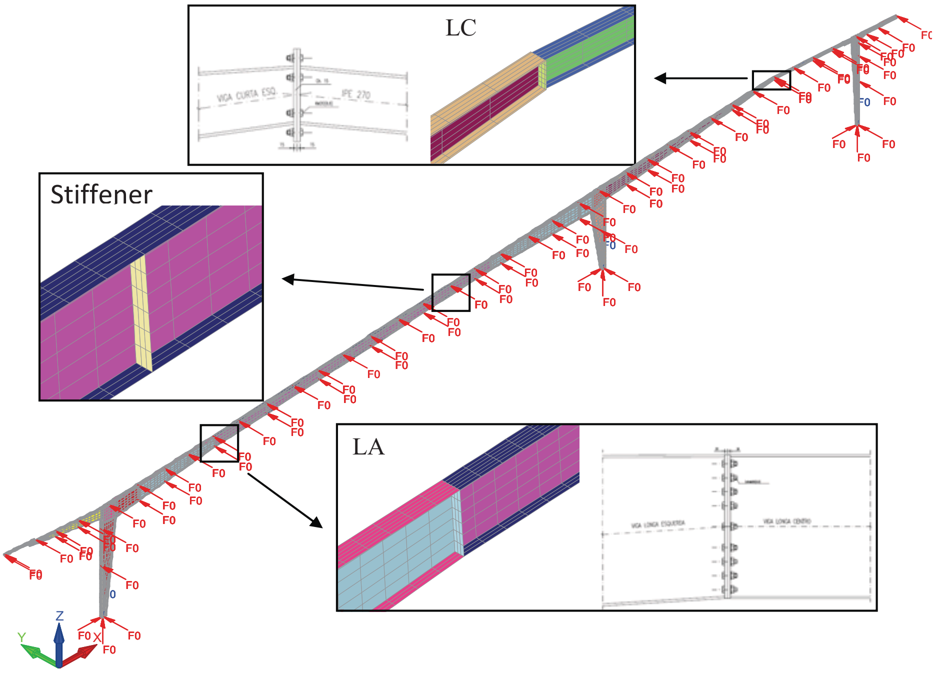



Fig. 14.8 Model restrictions and details of the beam to beam connections and of a transverse stiffener

Portal frame without insulation

On a first analysis it was considered that the portal frame did not have any fire protection, and that was subjected to the natural fire scenarios obtained from the thermal analysis (see Section 2). It must be mentioned that for the analysis of the portal frame subjected to natural fire curve in Zone A, and with actions combination 1, the collapse occurred at 41 minutes, being clear that fire protection is needed.

Portal frame insulated

Subsequently, the portal frame, insulated for 60 minutes of ISO834 considering a critical temperature of 500°C, was analysed, subjected to the natural fire scenarios obtained from the thermal analysis (see Section 2). For each of the fire scenarios (fire in Zone A, fire in Zone B and localized fire in Zone C) the three actions combinations, listed above, were considered.

**Fire in Zone A**

In structural analysis, it was considered that the fire in Zone A corresponds to having the beams V2, V3 and V4 heated and the other elements without any increasing temperature. There was no occurrence of structural collapse during all the fire, for the three actions combinations. The portal frame deformed shape, when subjected to the natural fire curve in Zone A and to the actions combination 1, at 120 minutes, is shown in Fig. 14.9.

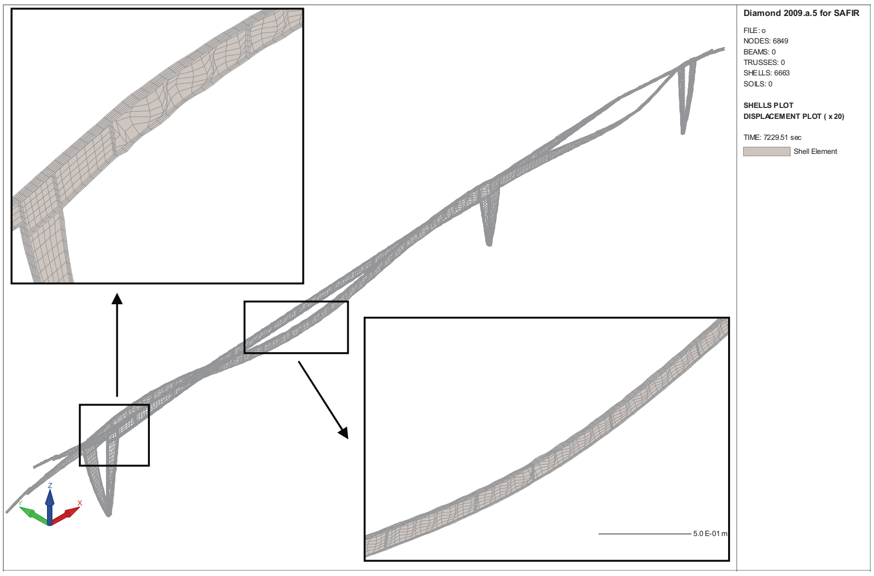



Fig. 14.9 The portal frame deformed shape, when subjected to the natural fire curve in Zone A and to the actions combination 1, at 120 minutes (x20)

**Fire in Zone B**

It was considered that the fire in Zone B corresponded to having only the column P2, the beam V5 and the IPE270 located between the columns P2 and P3 (see Fig. 14.4) heated. It was not observed structural collapse during all the fire, for the three actions combinations. The portal frame deformed shape, when subjected to the natural fire curve in Zone B and to the actions combination 1, at 120 minutes, is shown in Fig. 14.10.

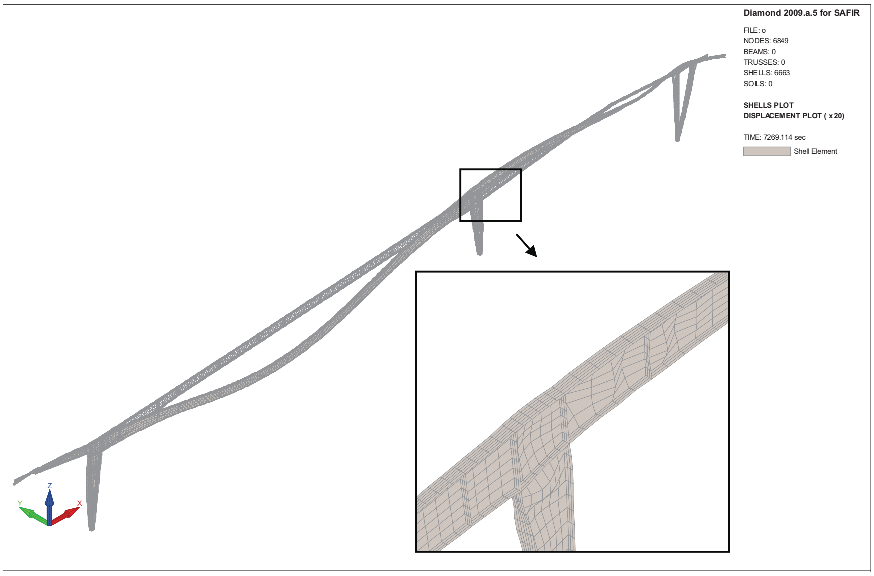



Fig. 14.10 The portal frame deformed shape, when subjected to the natural fire curve in Zone B and to the actions combination 1, at 120 minutes (x20)

**Localized fire in Zone C**

It was considered that the localized fire in Zone C would affect only column P1 (most severe case). As the temperature varies with the height along the axis of the flame (see Fig. 14.3), the columns was subdivided into 1 meter parts in the length, where it was applied, in which of them, the corresponded temperature evolution, as illustrated in Fig. 14.11.

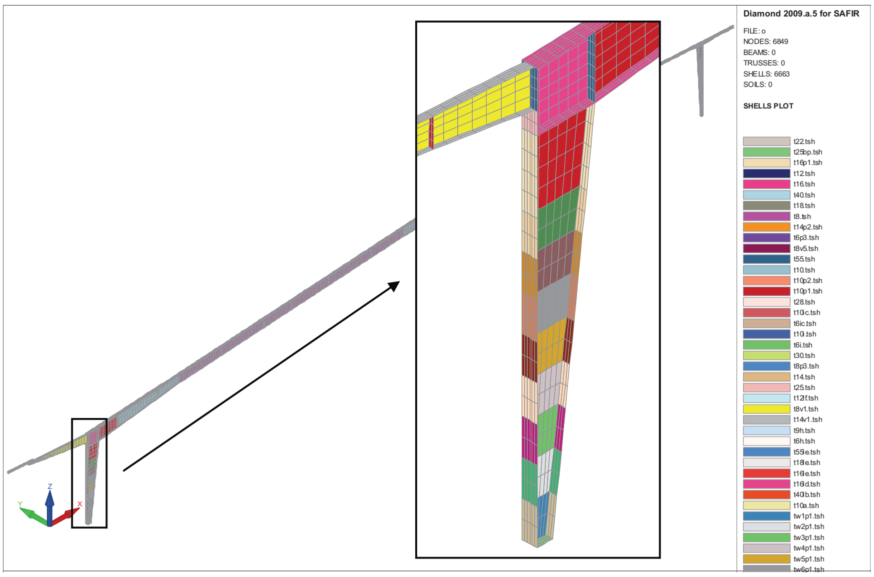



Fig. 14.11 Different temperature evolution in function of column P1 height for the localized fire

It was not observed structural collapse during all the fire, for the three actions combinations. The portal frame deformed shape, when subjected to the localized fire curve in Zone C and to the actions combination 1, at 120 minutes, is shown in Fig. 14.12.

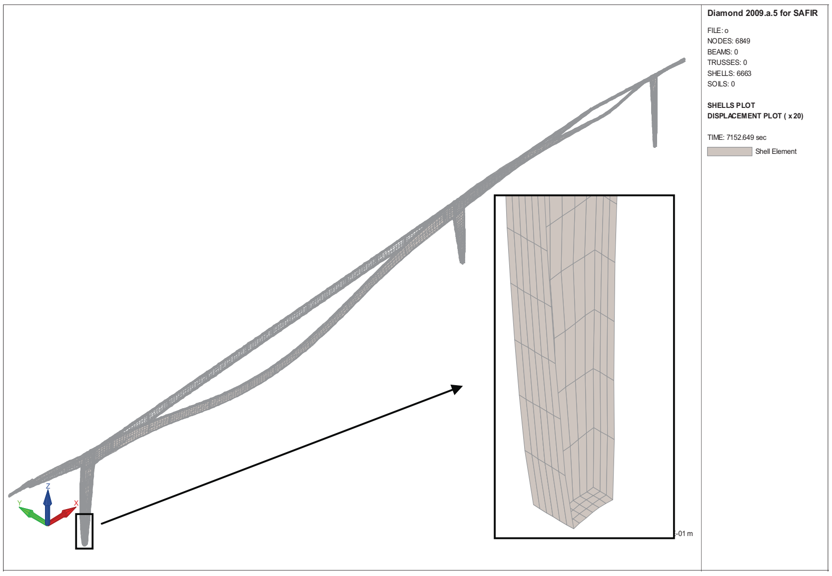



Fig. 14.12 The portal frame deformed shape, when subjected to the localized fire curve in Zone C and to the actions combination 1, at 120 minutes (x20)

Results

Tab. 14.1 summarizes the results of the described above analyses, indicating the collapse instance (tc) when this occurs.

Tab. 14.1 Results

| **Analysis** | Result |
|---|---|
| Unprotected portal frame subjected to natural fire in Zone A and the actions combination 1 | $t_c$ = 41 min |
| Protected portal frame: | |
| Natural fire in Zone A: | |
| Actions combination 1 | No collapse |
| Actions combination 2 | No collapse |
| Actions combination 3 | No collapse |
| Natural fire in Zone B: | |
| Actions combination 1 | No collapse |
| Actions combination 2 | No collapse |
| Actions combination 3 | No collapse |
| Localized fire in Zone C: | |
| Actions combination 1 | No collapse |
| Actions combination 2 | No collapse |
| Actions combination 3 | No collapse |

## 14.4 CONCLUSIONS

The steel structure of the Oeiras Exhibition Centre is composed of profiles with non-uniform class 4 cross-sections. A prescriptive analysis based on ISO834 standard fire curve, without evaluating the structural performance, would require the use of passive fire protection designed to provide a fire resistance of 120 minutes (R120) considering, according to EN 1993-1-2, a critical temperature of 350°C. This protection would be very expensive.

The Portuguese fire regulation states that: "Depending on its type, the buildings structural elements shall have a fire resistance to ensure their load bearing, thermal insulation and integrity functions during all stages of the fire fighting, or alternatively, must have the minimum standard fire resistance." In this case, the minimum fire resistance would be of 120 minutes.

A performance- based analysis has shown in this study that, without sacrificing safety, protecting the structure for a standard fire resistance of 60 minutes (R60), assuming a critical temperature of 500°C, is possible to maintain its load bearing functions during all phases of the fire, including the cooling phase.

It was proposed therefore that the main steel structure would be protected to R60 considering a critical temperature of 500°C.

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# 15 CAIRO EXPO CITY EXHIBITION CENTRE FIRE ENGINEERING

Summary

The Exhibition Centre at Cairo Expo City will comprise 4 large halls (in excess of 120m in dimension each) which will cater for very large numbers and will be accessed from a single continuous entrance foyer. Because of these features compliance with code guidance in relation to travel distances and exit capacity was not practical in the halls and entrance foyer. Therefore a comprehensive fire engineered solution was undertaken involving the provision of smoke extract to maintain tenable conditions during escape. This approach involved the use of CFD smoke modelling in combination with an analysis of escape times to ensure that occupants can escape before escape routes were rendered untenable. The CFD analysis was also used to inform a Structural Fire Engineering analysis of the space frame to ensure that premature collapse of the structure does not occur in a fire scenario. Outlined in this paper is a description of the approach adopted for both the escape and structural fire engineering aspects on this scheme.

## 15.1 INTRODUCTION

The exhibition centre at the new Cairo Expo City development comprises a curved roof structure sweeping over four internal exhibition halls and covering over 180,000m³ of plan area which will make it an exhibition hub in the Arab world. Each of the 4 rectangular column free halls is 120m wide and up to 360m long, and are all linked by a large continuous circulation zone/entrance lobby. Within the entrance lobby there are open balconies at first floor providing access to a large number of individual conference rooms. Buro Happold is the multi-disciplinary consulting engineer for the entire development, working with Zaha Hadid Architects.

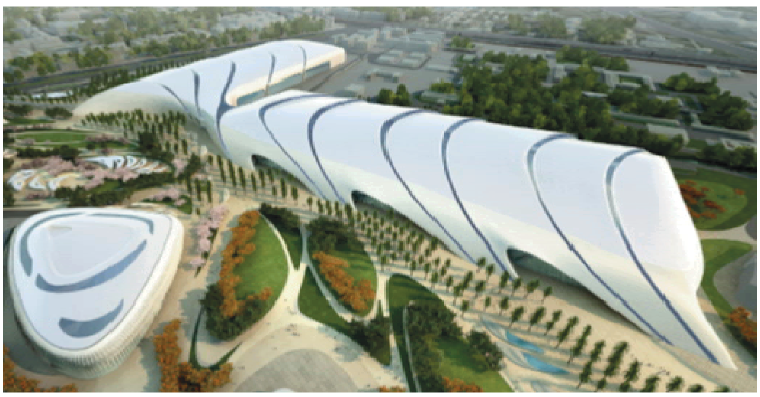


Fig. 15.1 Exhibition Centre – Exterior Render © Zaha Hadid Architects

Because of the sheer size of each of the halls and entrance lobby and the large numbers of expected occupants compliance with the relevant prescriptive codes NFPA 101 and Egyptian Building Code in terms of travel distances and exit capacity was not practical. The fire strategy for the Exhibition Centre does not follow the prescriptive requirements of NFPA or Egyptian Code as several key elements are too restrictive or do not result in a safe solution when taking the site constraints of the building into consideration:

- the maximum permitted 76m travel distance limit is not feasible without compromising the desired openness of the exhibition halls,
- using 5mm/p and a 50% limit for the horizontal exit width to the foyer would overload the fairly narrow rear service road to the West of the Exhibition Centre, potentially creating a conflict between emergency vehicle access and people evacuation,
- At least half of the stairs serving the upper level within the Exhibition Centre would have to exit to the outside to comply with code, which would require protected exit passageways within the foyer, again compromising the desired openness and connectivity of the foyer.

Therefore a project specific fire strategy tailored to the site constraints and client brief, which follows a fire engineered approach based on Chapter 5 of NFPA has been utilised. The assumptions and relevant characteristics for this assessment are outlined below. This case study concentrates on the fire engineered analysis for means of escape from the halls.

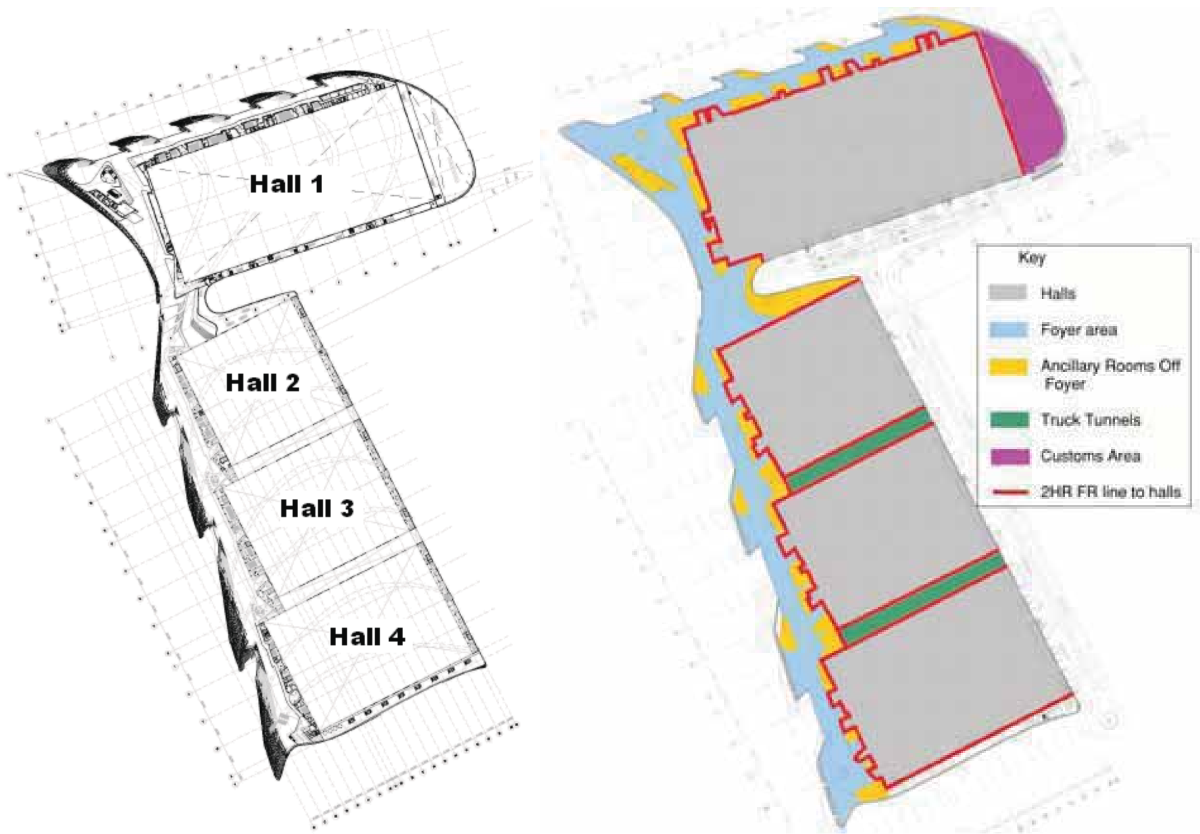



Fig. 15.2 Exhibition Centre Plan and Evacuation Zones

## 15.2 FIRE STRATEGY PRINCIPALS

The main principal was to split the Exhibition Centre into separate evacuation zones/compartments whereby only the zone on fire will be evacuated initially. The Foyer and each of Halls 1 to 4 each form separate evacuation zones. This approach was chosen to limit the numbers of occupants escaping at any one time in order to enable more effective and easier evacuation management and more efficient escape design. This also enabled each of the halls and foyer to be assessed individually.

The Exhibition Centre will be fitted with sprinklers and smoke control systems, automatic smoke detection for early notification, which together with management requirements form an essential part of the fire safety strategy.

## 15.3 PERFORMANCE BASED APPROACH PROCESS

There are four key processes to the fire engineering as described below. These components also relate to the NFPA frame work for performance-based options.

Tab. 15.1 Outline of the processes to the Fire Engineering Brief (FEB)

| Process | |
|---|---|
| Define and agree objectives and criteria | The fire safety goals and objectives are defined and agreed amongst all stakeholders at earliest stage possible. These include regulatory requirements and client and insurance requirements for property protection and business continuity as well as Civil Defence requirements. |
| Design specification – part 1 | This includes a brief description of the buildings, the assumptions made, hazard identification, retained prescriptive requirements, fire scenarios (including occupant characteristics, design fires and proposed systems and response). |
| Design specification - part 2 | The detail of the fire strategy covering proposed systems and management assumptions will be documented in the Fire Strategy Report. The strategy outlines the initial trial design. |
| Define and agree performance assessment | The acceptance criteria, assessment methods and tools, safety factors and output are defined and agreed that will be used in the analysis. Once a trial design has been agreed and shown to be acceptable, it becomes the Proposed Design. |
| Define and agree outputs and documentation | Effective and clear documentation is an important part of the fire engineering process. The output provided from the analysis and the final documentation are also agreed in the Fire Engineering Brief upfront. |

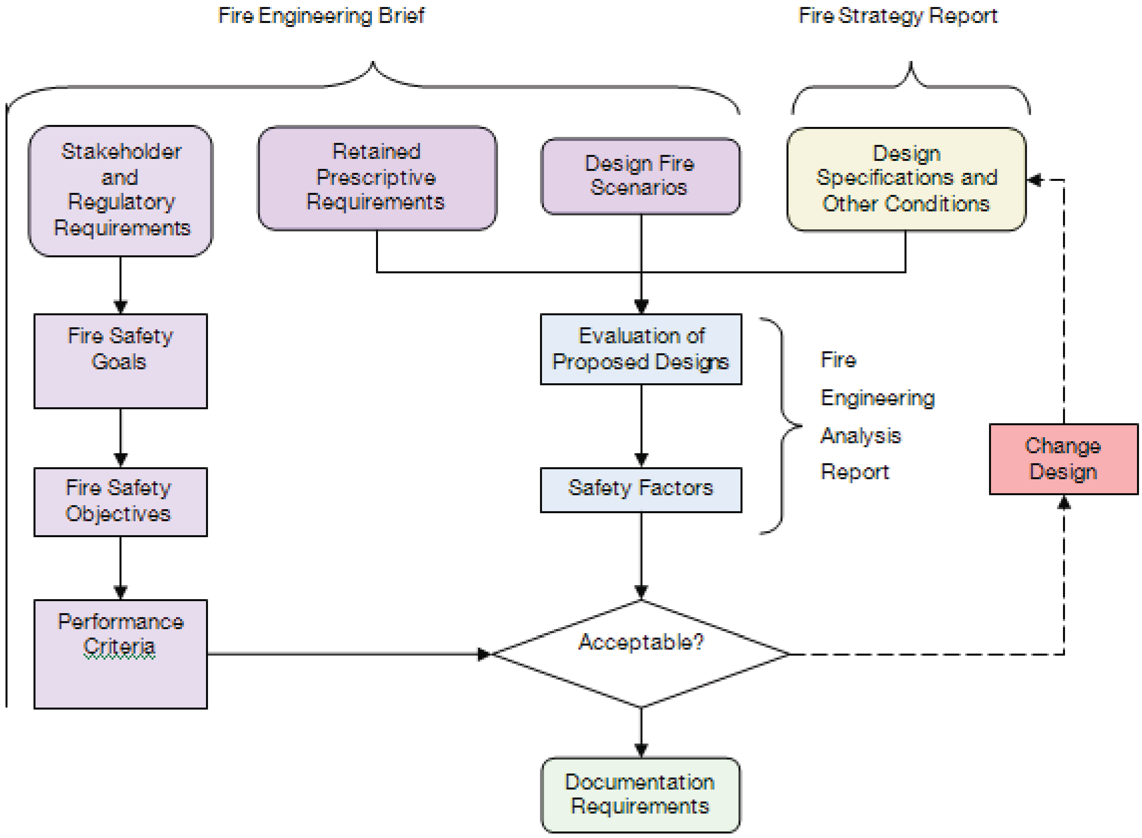



Fig. 15.3 Performance Based Life Safety Code Compliance Process

**15.4 ASSESSMENT STRATEGY AND ACCEPTABILITY CRITERIA**

A detailed assessment was carried out to demonstrate that the smoke control provisions are sufficient to ensure that all occupants can evacuate to a place of safety before the condition in the building becomes untenable by way of comparing the Available Safe Egress Time (ASET) with the Required Safe Egress Time (RSET), and applying a suitable margin of safety as shown below.

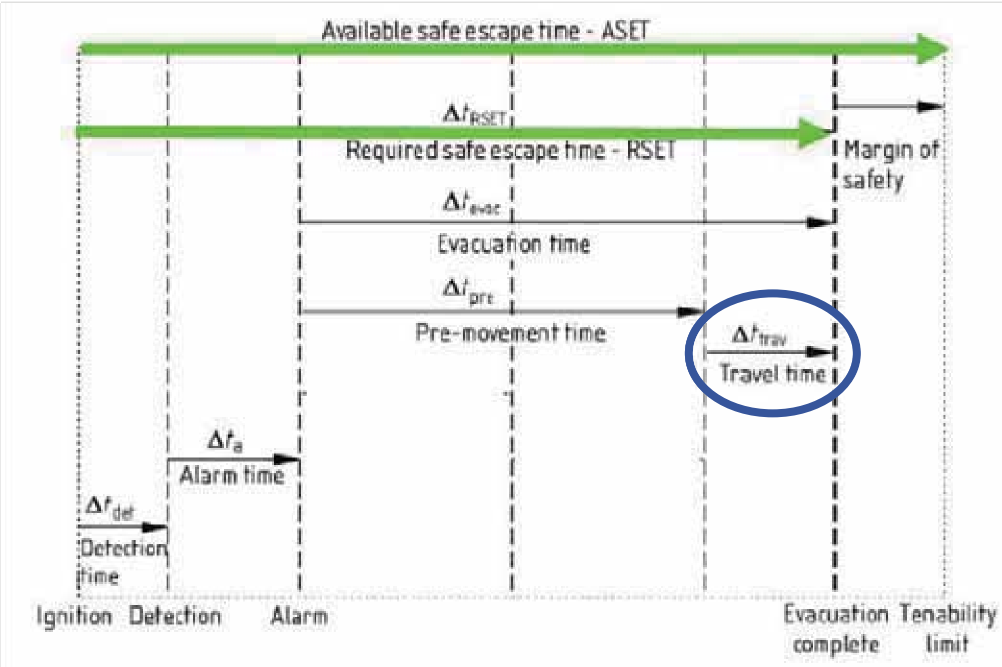



Fig 15.4 ASET - RSET Design Approach

The design further ensures for Crowd Comfort that the movement or travel time, i.e. the time taken to walk to a place of interim safety which is highlighted in the figure above by a blue circle, does not exceed 6 minutes. The acceptance criteria for the assessment are outlined below.

Tab. 15.2 Acceptance Criteria for Fire Engineering Analysis

| | **Value** | **Reference & Comments** |
|---|---|---|
| Hot Layer depth | maintain smoke layer at least 1.83m (measured to 2m within the CFD analysis) above floor level | applies to zone model calculations and CFD analysis (NFPA 92B) |
| Temperature | if smoke layer descends closer than 2.1 metres above floor then smoke temperature is not to exceed 60 deg C | exposure for no more than 10 minutes, measured 1.83m above floor level (NFPA 92B, NFPA 130) |
| Visibility | Optical density not to exceed 0.1 m-1 (i.e. 10 metres visibility). Refer note 1 | applies on escape routes for the required safe egress time, measured 1.83m above floor level |
| Toxicity | Exposure to toxic and irritant products is not considered separately. Criteria for visibility are used to control limits on exposure to smoke and toxic products as both are a function of smoke production and dilution | Applies to CFD and zone models at early stages of a fire. Does not apply during fire fighting. Refer Wade in Chapter 8 of FEDG, Buchanan, 2001 |
| Radiant Heat | for smoke more than 1.83 metres above floor, radiation is not to exceed 2.5 kW/m2 (typically 200 deg C). Refer Note 2 | measured at 1.83m above floor; applies to zone model calculations only. NFPA 130 |

## 15.5 DESIGN FIRE SIZE

A number of Design Fires were considered as outlined below:

Tab 15.3 Design Fires Considered

| Design Fire | Scenario | Design Fire Characteristics |
|---|---|---|
| 1 | Occupancy specific, accounting for the occupant activities, furnishings and room contents, fuel properties, ignition sources and ventilation conditions. | 1.1 Fire in multi-storey exhibition booth, sprinkler controlled. Max HRR 1MW, height of fire at floor level, slow growth.<br>1.2 Fire in multi-store exhibition booth, not sprinkler controlled. Fire at top level, 9m above floor level or at floor level, max HRR 6MW, medium growth.<br>**1.3 Fire at floor level, by small car or art exhibit, with maximum HRR of 6MW, and fast fire growth.** |
| 2 | Ultrafast developing fire in primary means of egress, with interior doors open at start of fire. | Fire at floor level, by small car or art exhibit, with maximum HRR of 6MW; fire growth initially ultrafast to 1.1MW, then medium fire growth. |
| 3 | Fire start in normally unoccupied room, potentially putting large numbers of people at risk in adjacent space. | Building is fitted with automatic smoke detection and sprinklers throughout, and adjacent spaces are fire separated from exhibition halls. This scenario is therefore adequately addressed by qualitative analysis and does not require further assessment. |
| 4 | Fire start in a concealed wall or ceiling adjacent to large occupied room. | Refer above – spaces are fire separated and fitted with automatic detection & sprinklers. |
| 5 | Slowly developing fire, shielded from fire protection, in close proximity to high occupancy area. | Building is fitted with automatic smoke detection for early notification. Analogue addressable system will allow exact identification of location and timely intervention by trained staff / fire service. Halls are fire separated from surrounding areas. |
| 6 | Most severe fire resulting from largest possible fuel load characteristic of the normal operation of the building. | **Fast growing very large fire, representing a large exhibition item such as a truck on fire. $Q_{max}$= 30MW (unless controlled earlier due to sprinkler activation or fire service intervention). Height of fire at floor level, fast fire growth.** |

Based on the above, two worst case fire scenarios were chosen as the design basis:

- a fast growing fire up to 30MW representative of a truck fire; and
- a fast growing retail exhibit fire up to 6MW. This was chosen to ensure that the smoke control system was also capable of dealing with a smaller fire which would result in reduced some temperatures but a less buoyant smoke layer.

Due to the high ceilings both fire sizes have assume that sprinklers do not act sufficiently early to control the size of the fires.

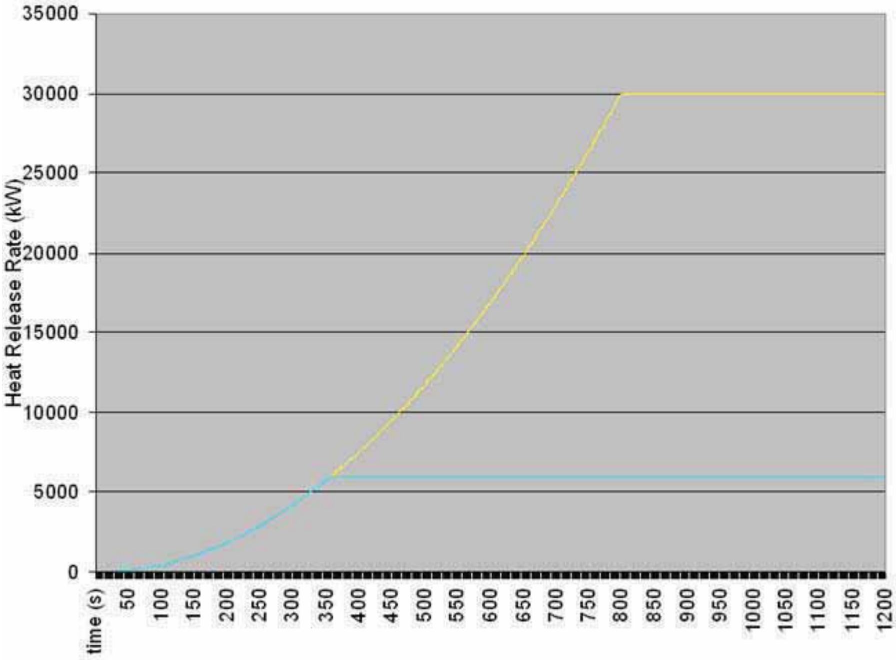



Fig. 15.5 Graph of HRR vs Time for Chosen Design Fires

**15.6 SMOKE CONTROL METHODOLOGY**

Halls 2 to 4 were divided into at least two smoke reservoirs by the inherent hall subdividing dividing downstand feature in the centre of each hall. Similarly Hall 1, due to its size was subdivided twice to form three separate reservoirs. Due to the hot climate in Cairo, a natural smoke extract system would not be suitable as it relies on the buoyancy of the smoke for venting. Therefore a mechanical smoke extract system was chosen with at least twelve extract points located throughout the roof of each reservoir. Inlet air to the system is provided by automatically opening the large service entrance shutters on the external facade.

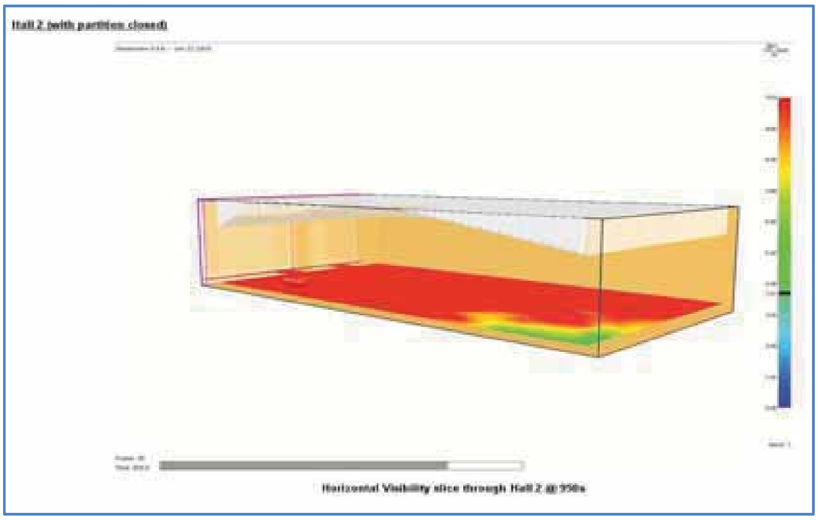
The CIBSE Guide E: Fire Engineering zone model approach was used to calculate the required extract rates for the above design fires. Due to the unusually large reservoir sizes it was necessary to carry out a CFD analysis of the smoke modelling to ensure that the system was capable of maintaining tenable conditions during escape.

**15.7 CFD ANALYSIS TO VERIFY AVAILABLE SAFE ESCAPE TIME (ASET)**

A number of CFD models were run for both a 30MW and 6MW fast growing fires using NISTs FDS program. The fires were modelled in a number of different locations to determine the worst case scenario, for example:

- Both remote and in close proximity to the inlet air openings
- In Hall 2 typical smallest reservoir, and Hall 4 typical largest smoke reservoir

A sensitivity analysis was also carried out for a medium growth rate.



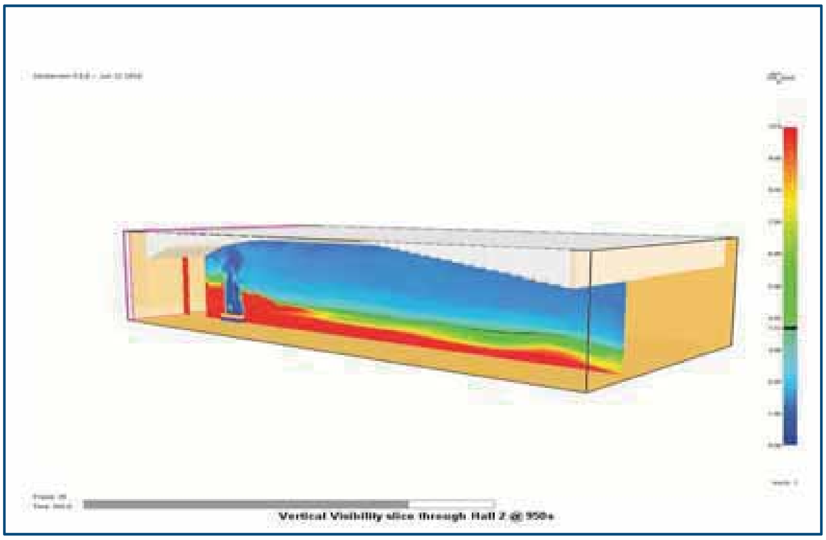




Fig. 15.6 CFD Model Slices of visibility at 950s in typical Worst Case Scenario at Tenability Limit

### 15.8 ASET VS RSET ANALYSIS & RESULTS

The CFD analysis has demonstrated that in the worst case scenario, visibility was the limiting factor on ASET with visibility being reduced below 10m at head height in one corner of Hall 2 after 950 seconds (almost 16 minutes). Therefore 950s has been taken as the benchmark ASET. It should be noted that this is considered a conservative approach given the following:

- This worst case scenario is for a 30MW fast growing fire which is intended to represent a very large display fire (e.g. a bus, boat or truck or similar sized display) and ignores the effects of sprinklers on the fire growth and size.
- Even with this conservative fire size, after 950s, the visibility is only reduced in one corner of the hall, the remainder of the hall remains within tenable limits.
- It is highly unlikely that the halls will be occupied to a level of 1.4sqm/person at the same time there is very large displays such as boats trucks etc. present.

The goal is to achieve an available safe egress time that exceeds the required safe egress time:

**ASET > = RSET**

The calculation of the required safe egress time (RSET) contains adequate safety factors to compensate for uncertainties within the evacuation assessment such that no further safety margin is required in the assessment. The results are summarised below for the typical worst case Hall which an occupant load of 20160 occupants.

When considering an extended pre-movement time (which has been set to 90 second, based on BS 7974) and alarm activation time and after incorporating a safety factor of 1.5 on the travel time, the last person is expected to leave after 10.4 minutes, with a safety margin of at least 5.4 minutes to the safe egress time. This 10.4 minute evacuation time also incorporates an additional factor of safety as the travel time has been increased by a factor of safety of 1.5. The flow and travel times are assessed based on the recommendations of NFPA130.

Tab. 15.4 Calculation of Available Safe Egress Time

| Description of Event | Time [sec] | Time [min] |
|---|---|---|
| Fire starts | 0 | 0.0 |
| Fire is detected (double knock activation) | 70 | 1.2 |
| Evacuation signal is raised immediately after double knock | 70 | 1.2 |
| First person starts to move | 160 | 2.7 |
| First person reaches exit | 248 | 4.1 |
| *Fire Service begins to fight the fire* | *630* | *10.5* |
| Last person leaves the hall | 470 | 7.8 |
| .. - with safety factor | 625 | 10.4 |
| Available Safe Egress Time (worst case Hall 2 used as benchmark ASET) | **950** | **15.8** |


Fig. 15.7 Timeline of Fire in Hall Worst Case Scenario

## 15.9 CONCLUSION

Due to the large open spaces required to meet the desired functional and architectural aspirations for the Exhibition Centre, compliance with prescriptive code was not possible in terms of travel distance and exit capacity. It was therefore necessary to use an alternative fire engineered approach whereby smoke control was used to maintain tenable conditions during escape for a worst case credible fire scenario. A CFD analysis was carried out for a number of different credible worst case fire scenarios to demonstrate that the smoke control system was capable of keeping escape routes tenable for at least 16 minutes (ASET). This is significantly greater than the estimated required safe escape time of 10.4 minutes (RSET). The CFD analysis was also used to inform a Structural Fire Engineering analysis of the space frame to demonstrate that premature collapse of the structure does not occur in a fire scenario. This case study has clearly demonstrated that the latest sound fire engineering techniques can be used as a powerful tool to ensure that a high level of safety can be achieved without limiting the flexibility in design of large and complex public buildings by prescriptive code approaches.

Acknowledgement

The authors would like to thank Zaha Hadid Architects for their permission to present this work.

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# 16 FIRE SAFETY ENGINEERING IN AN AIRCRAFT HANGAR (DEFINITION OF FIRE SCENARIOS AND DESIGN FIRES)

Summary

On the premises of the new airport of the city of Berlin, a line maintenance hangar for aircrafts was planned and will be finished until the second half of 2012. The fire safety consultants hhpberlin developed the relevant fire scenarios and defined design fires as a basis for CFD simulations. The possibility of successful fire fighting measures and the design of the load-bearing steel structure of the roof and the upper parts of the columns without protective coating were investigated on the basis of results of the simulations.

## 16.1 DESCRIPTION OF THE CONSTRUCTION

The hangar has the outer dimensions of 83.40 m width and 77.60 m depth and thus an inner area of approx. 6,472 square meters. The hangar has a horizontal orientated roof, which is supported by an external two truss girders on steel supports that are located on the outside. A secondary internal load-bearing system is attached below the main load-bearing elements. The hangar has a medium interior height of approx. 18.10 m, the height of the lower edge of the secondary load bearing structure is 15.25 m.

## 16.2 SAFETY OBJECTIVES

The building was designed on the basis of the German guidelines for industrial constructions. The first objective that has to be fulfilled is the sufficient structural safety in case of fire. The second objective for buildings that have an area of more than 1,600 square-metres is the proof of the possibility of successful fire-fighting.

In Germany this safety objective is achieved in case a smoke layer height of at least 2.5 m is verified for every level of the building.

In the present aircraft hangar this has to be differentiated more precisely, because for effective fire-fighting at least the fuselage of the aircraft parked and maintained in the hangar must be recognizable and must reside outside the smoke layer. The smoke-exhaust measures must be designed to keep the top edge of the highest aircraft out of the smoke layer.

The present case involves the Boing B747, which has a maximum height of approx. 10 m at the top of the cabin. The minimum height of the smoke layer to account for is thus 10 m.

The safety objective of ensuring a smoke layer height was fulfilled in a CFD simulation. As criteria to prove the safety objective the so-called optical smoke density is evaluated. It can be assumed that the safety objective is met when the optical smoke density does not exceed a value of 0.15 1/m (vfdb Leitfaden 2009).

In addition to the use of the results from the simulation for the design of the smoke exhaust systems, they are used for the structural fire design of the steel structure according to the simplified calculation procedures of (Eurocode 3) (columns and roof structure).

## 16.3 FIRE SCENARIOS

For the proof of the safety objectives all the relevant fire scenarios must be investigated.

Various aircrafts are to be serviced in the hangar. To demonstrate the stability of the load bearing structure of the roof, it is the worst case to assume that the fire occurs as high as possible in the building. To prove the effectiveness of the smoke exhaust measures for successful fire fighting operations, the amount of smoke gases produced by a fire at the lowest point in the building is relevant, because the longer the distance on which fresh air is entrained into the plume, the greater is the quantity of produced smoke gas.

Because of the wingspan of the aircrafts and the specified distance of the aircraft from the supporting structure there will not be high heat release rates from a fire close to the load bearing coloums. It cannot be excluded that a fire erupts directly near the supports of the roof structure, for example due to fires of technical equipment or temporarily positioned storage goods. For this reason another fire scenario 3 ("local fire") must be investigated in addition to fire scenarios 1 and 2 ("aircraft fires"). This fire is considered examplarily in the positions of relevant coloums of the main supporting structure.

The largest aircraft in the hangar and thus the potentially largest origin of a fire is the Boeing B747. The aircraft measures 71 metres in length and has a wing span of about 69 m, as well as a body diameter of around 7,30 m at the highest point, with a height above the floor of approx. 2.70 m.

In principle fire events have to be expected in the disadvantageous place in the building. If a fire occurs in a corner area of the building the temperatures it causes are typically higher due to accumulation effects. Due to the fact that the hangar doors at the front of the building are opened in the event of a fire, positions at the back of the building are viewed as disadvantagues.

The aircraft (B747), which is considered most relevant for assessment, has two levels. In the front section of the Boeing B747 an upper deck is located. It is therefore no longer apparent where a design fire will lead to higher temperatures in the building or at the bottom edge of the roof structure. Therefore two aircraft fire scenarios (see Fig. 16.1) were adopted, where the place of the fire was assumed at the back corner of the hangar in both cases.

- Scenario 1: fire in the cabin of a B747-400 with participation of part of the wings (plastics and kerosene fire), fire surface 100 m², fires in a height of approx. 6 m.
- Scenario 2: cabin fire in a B747-400 in the upper-deck (plastics) without participation of the wings, fire area 50 m², fires in a height of approx. 8 m.
- Scenario 3: local fire, for example larger car or a similar major technical device or storage good, fire area 10 m², fires at a height of 4.0 m.

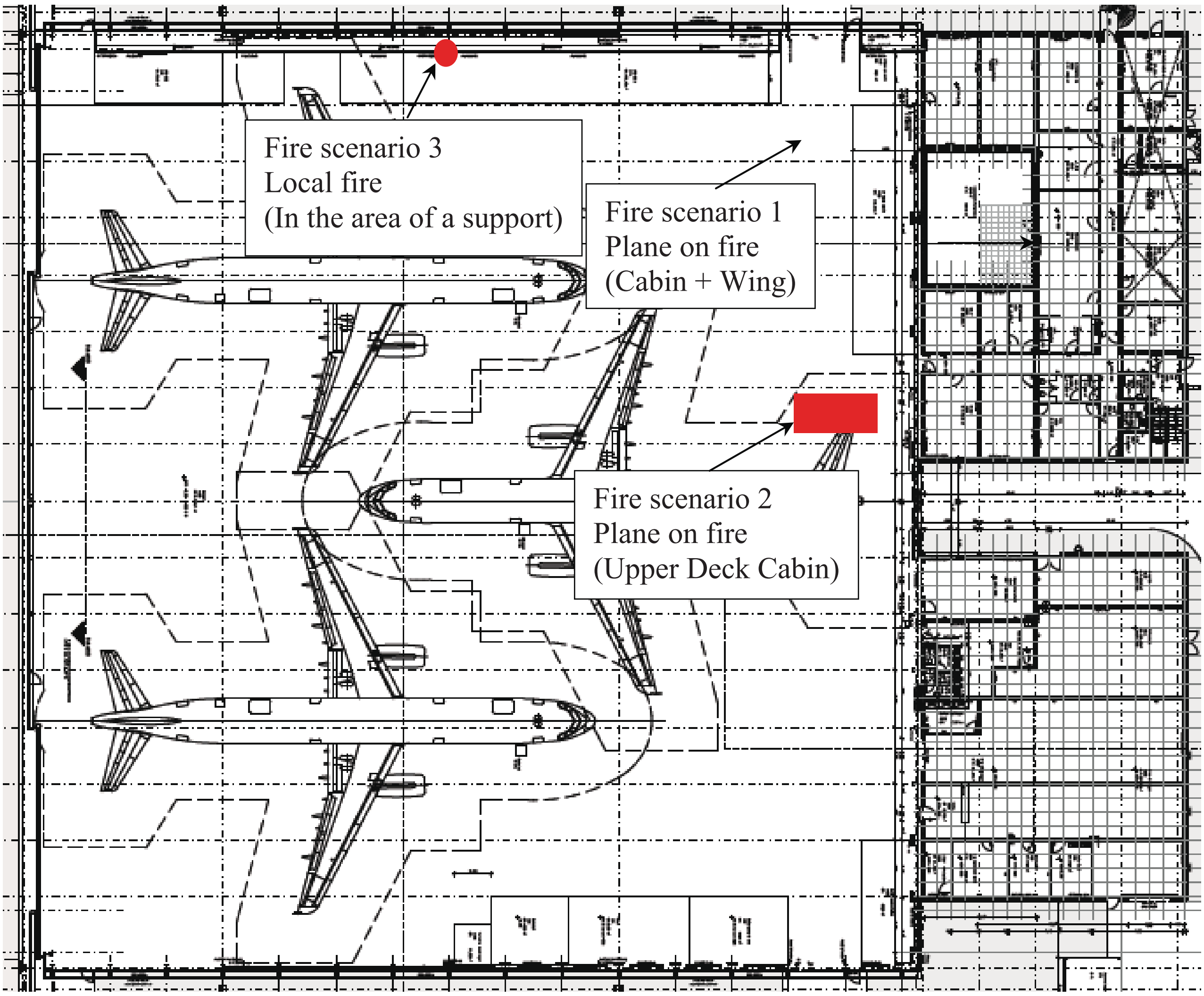


Fig. 16.1 Floor plan of the hangar with the position of the origin of the fire

## 16.4 DESIGN FIRES

To ensure design fires on the safe side, the relevant input parameters must be adopted conservatively, so that all relevant fire events are covered.

With regard to the course of the heat release rate, it is assumed that the fire does not extinguish after the depletion of the fire load but will remain at the maximum rate of the heat release. For that reason, only the nature of the fire load, but not the amount is significant the for the fire safety design.

The safety objectives in the building will be proven when a stationary state, i.e. a balance of the energy supplied by the fire and the energy dissipated by the smoke and heat exhaust measures is reached. In all fire scenarios extinguishing measures, for example, by the extinguishing system or the airport fire brigade, are not considered to have a direct effect on the heat release rate. They are accounted for conservatively by the partial factor $\gamma_{fi,HRR}$, according to (DIN EN 1991-1-2/NA).

Due to the fact that a fire develops higher temperatures if it burns with a high heat release rate concentrated in a small area and on the other hand a higher smoke gas production occurs in a larger fire area, the realistic fire areas mentioned above are chosen for the aircraft fire scenarios.

In the following the curve progressions of the heat release rates are defined for all 3 fire scenarios. The maximum of the heat release rate $Q'_{max}$ is dependent on the speed of the fire development or the resulting fire area $A_f$ , as well as the area-based heat release rate of q'. Additionally a partial safety factor to account for fire-fighting measures according to the safety concept of the Eurocode (DIN EN 1991-1-2) is considered.

$$Q'_{max}\,[kW] = q'[kW/m^2] * A_f\,[m^2] * \gamma_{fi,HRR} \quad (1)$$

According to the literature such as the (vfdb Leitfaden), area-specific heat release rates of between 150 kW/m$^2$ and 500 kW/m$^2$ are realistic. In individual cases also values over 600 kW/m$^2$ can occur especially in plastics and lubricants. This adds up to the following design fires for the defined fire scenarios (partial safety factors in accordance with DIN EN 1991-1-2 / NA):

- Scenario 1 (Fire in cabin + wing): q' = 600 kW/m²

  $$Q'_{max}\,[kW] = 600\,kW/m^2 * 100\,m^2 * 1{,}075 = 64500\,kW \text{ after approx. } 930\,s \quad (2)$$

- Scenario 2 (Fire in upper-deck cabin): q'=450 kW/m²

  $$Q'_{max}\,[kW] = 450\,kW/m^2 * 50\,m^2 * 1{,}075 = 24187.5\,kW \text{ after approx. } 720\,s \quad (3)$$

- Scenario 3 (Local fire at support): q'=500 kW/m²

  $$Q'_{max}\,[kW] = 500\,kW/m^2 * 10\,m^2 * 1{,}075 = 5375\,kW \quad (4)$$

The illustrations in Fig. 16.2, Fig. 16.3 and Fig. 16.4 show the curve progressions of the heat release over time.

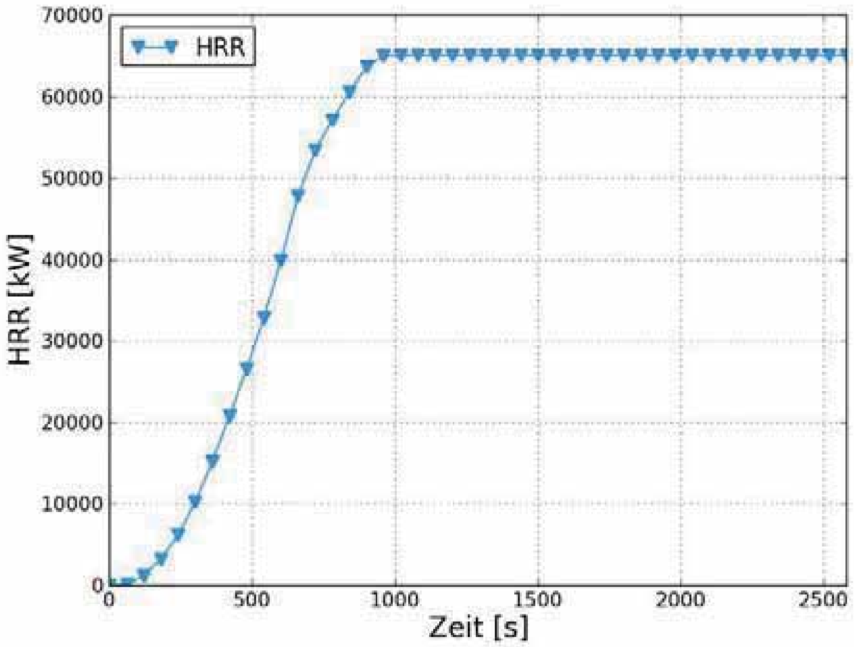


Fig. 16.2 Chronological sequence of heat release rate (HRR) in the fire scenario 1 ("aircraft fire")

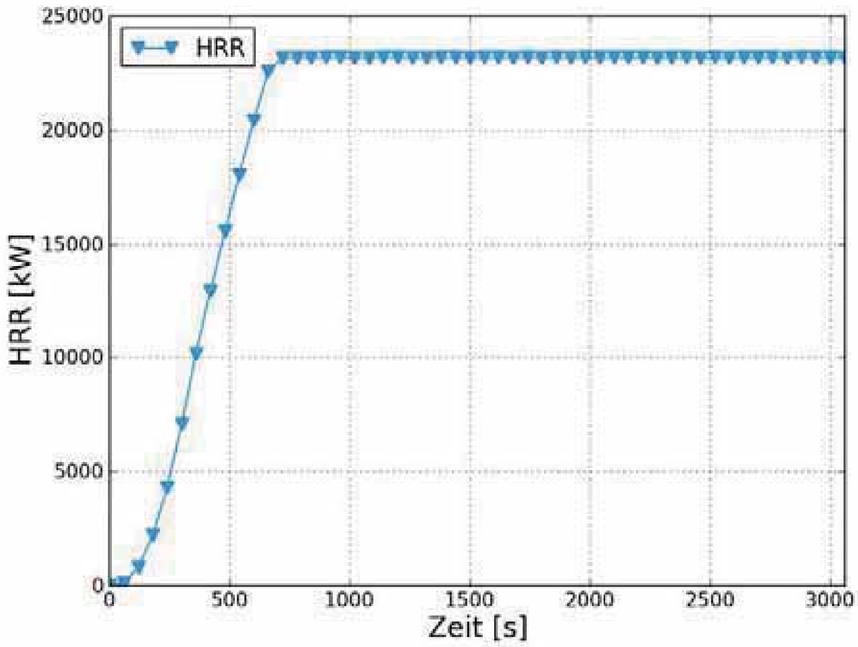


Fig. 16.3 Chronological sequence of heat release rate (HRR) in the fire scenario 2 ("Fire in upper deck cabin ")

A rapid fire propagation speed is assumed, so that a heat release rate of 1 MW is achieved after 150 s (see Fig. 1.4).

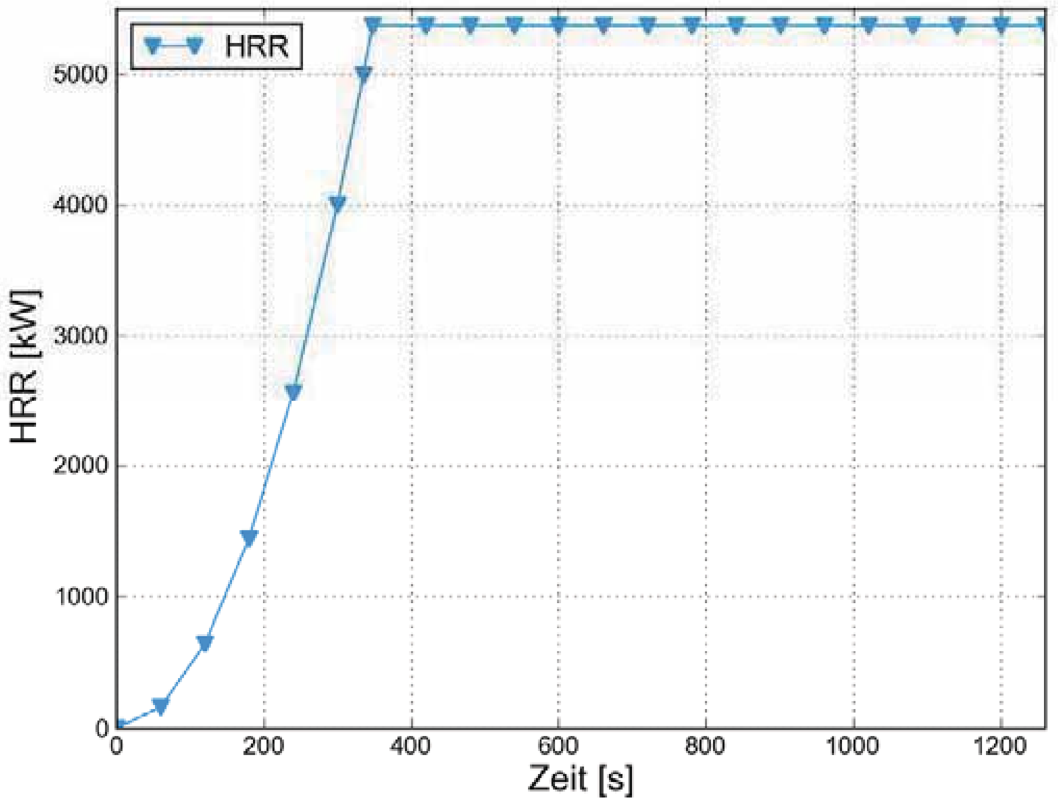


Fig. 16.4 Chronological sequence of heat release rate (HRR) in the fire scenario 3 ("Local fire ")

The specification of the source of a fire is required for fire simulations. The calculations are based on a mixed load of fire, which consists of 33% wood, 33% polystyrene, and 33% polyurethane and thus is based by 2/3 on plastics. In the light of the existing flammable substances in the hangar this is to be regarded as conservative.

The mean value of the heat of combustion of the fuel mixture was derived from the specific parameters of the individual substances to be approximately 27 MJ/kg.

Due to the relation between the heat release q‘, the mass flow m‘ and the heat of combustion H (m‘ = q‘/H) a higher heat of combustion leads to a lower mass flow in case a heat release rate was specified. Thus this leads to a lower production of smoke gas, so that this value specified is considered to be conservative.

The soot yield was set to an average value of 0.09 g / g. This definition of soot production was derived from the information in the (vfdb Leitfaden 2009) and is adopted conservatively.

## 16.5 CONCLUSIONS

The case study discusses the relevant design fires and fire scenarios for an appropriate design of structural fire protection measures of the load bearing structure of the aircraft hangar. The described design fires and fire scenarios were the basis of simulations carried out to derive a temporal and spatial temperature distribution in the hangar as input into the fire safety design methods of Eurocode 3 part 1-2.

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# 17 PRODUCTION AND STORAGE HALL OBJECT

Summary

The main objective of the case study is to examine fire resistance of steel structure of production and storage hall object located in Mnichovo Hradiště, Central Bohemia, Czech Republic. The investigated part of the object is of irregular ground plan. Its maximal dimensions are 90.0 x 114.0 m, 13.25 m height. Inside the hall there is the area for production of lemonade and area for storage of the product. The structure is composed of precast concrete columns and steel trusses of 22.5 m length. Roof deck is designed from trapezoidal sheet, vertical cladding from sandwich panels. The requirement for the fire resistance of the bearing structure is REI 15. The fire resistance of steel trusses, stiffeners and other members of roof deck and vertical cladding are investigated. Regarding large dimensions of the hall as one compartment localised fires of wooden pallets and high-lift truck are considered.

## 17.1 DESCRIPTION OF THE HALL

The object of production and storage hall of lemonade is located in Mnichovo Hradiště, Central Bohemia, Czech Republic. The subject of the study is only part of the object which is of irregular ground plan. Its maximal dimensions are 90.0 x 114.0 m, height of the attic is 13.25 m and roof declension of 2 %. The one floor hall includes an area of production of lemonade and area for storage of the product. The structure is composed of precast concrete columns and steel trusses of 22.5 and 20.0 m length, simply supported on the head of concrete columns, see Fig. 17.1. The span length between steel trusses is 19.0 m. Both, steel trusses and steel truss purlins consist of H-section upper chord and lower chord, diagonals are from square tubular section. In roof plane there are stiffeners of round tube section. Roof deck is designed from galvanized trapezoidal sheet S.A.B. 150/280/0.75 as two-span continuous beam 2 x 5.625 m. Vertical cladding of the hall consists of sandwich panels on intermediate columns from H-section and vertical beams of 5.625 m span length.

## 17.2 FIRE ENGINEERING APPROACH

### 17.2.1 Subject of examination of fire resistance

The objective of the case study is to investigate fire resistance of steel trusses (steel truss P1 is shown at Fig. 17.2), steel stiffeners and other members of the roof, bearing members of cladding as intermediate columns (see Fig. 17.3) and horizontal beam above the portal door. The fire resistance of concrete columns, roof cover and sandwich panels is not subject of the case study. The requirement for the fire resistance of the main bearing structure is REI 15.

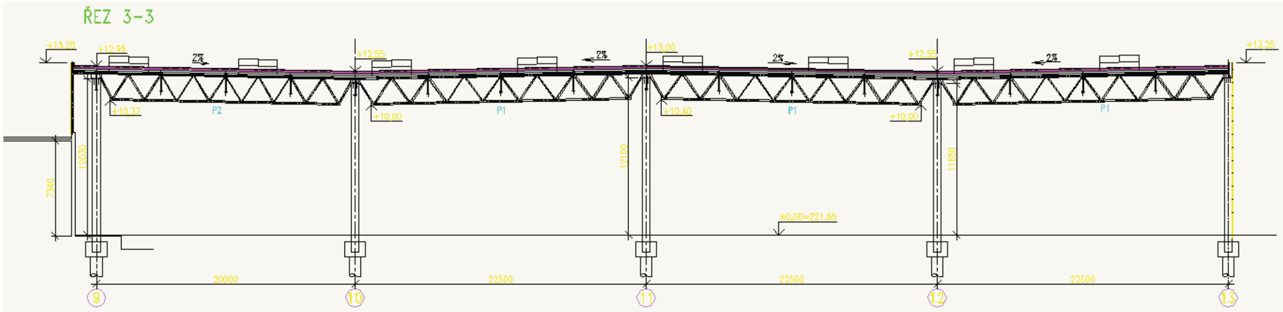


Fig. 17.1 Steel trusses of 22,5 m length

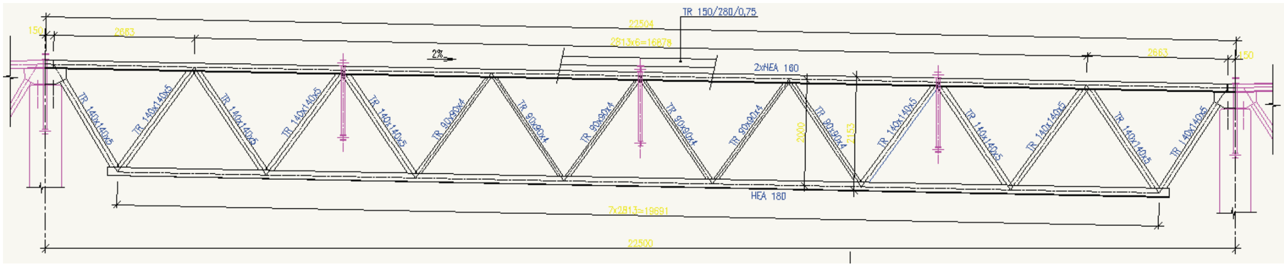


Fig. 17.2 Steel truss P1

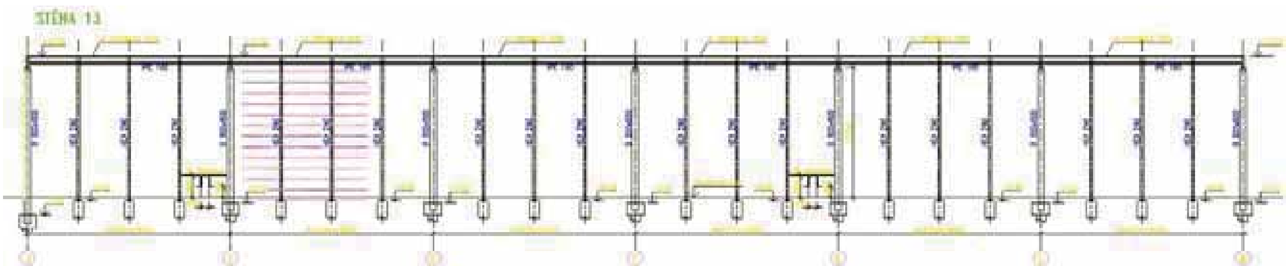


Fig. 17.3 Bearing members of cladding

### 17.2.2 Process of examination of fire resistance

The examination of fire resistance of steel structure covers several steps:

1. Selection of design fires
2. Description of design fires
3. Heat transfer analysis
4. Mechanical response analysis

## 17.3 DESIGN FIRE SCENARIOS

Regarding large dimensions of the hall as one compartment (area of 13 234 m$^2$) fully developed fire is not expected. Two localised fires are considered:

1. Localised fire of wooden pallets - product is stored on wooden pallets of weight 138 000 kg on area of 9 259 m$^2$. Fire load of 14.9 kg/m$^2$ is considered.
2. Localised fire of high-lift truck

Localised fire parameters are assessed by using the expressions given in EN 1991-1-2:2002 Annex C. By calculation of influence of localised fire to roof members it is assumed that the centre of fire is directly under the examined member. While by examination of intermediate cladding columns the method of heat transfer by radiation is used.

### 17.3.1 Localised fire of wooden pallets

The design value of the fire load is defined using EN 1991-1-2:2002 Annex E:

$$q_{f,d} = 577{,}2\, MJ/m^2$$

where characteristic fire load density per load area is $q_{f,k} = 260{,}8\, MJ/m^2$

factor $\delta_{q1} = 2{,}12$

factor $\delta_{q2} = 1{,}00$

sum of factors $\prod \delta_{n,i} = 1{,}305$ (no automatic water extinguishing system, no independent water supplies, installed automatic heat detection, no automatic smoke detection, no automatic alarm transmission to fire brigade, no work fire brigade, off site fire brigade, safe access routes are designed, fire fighting device are available, no smoke exhaust system)

To calculate time dependant rate of heat release $RHR_f$, the time needed to reach a rate of heat release of 1 MW is used as $t_\alpha = 300\, s$, $RHR_f$ = 1250 kW/m$^2$ (wooden pallets of 0.5 m height)and the fire diameter D = 3 m. The maximum heat release of wooden pallets is 8836 kW. The rising phase takes 14 min 52 s and stationary phase 28 s. Fuel burns out in 19 min 59 s, see Fig. 17.4. The length of the flame is calculated according to EN 1991-1-2:2002 Annex C. Fig. 17.5 shows the maximum length of the flame of 5.83 m (the flame is not impacting the ceiling).

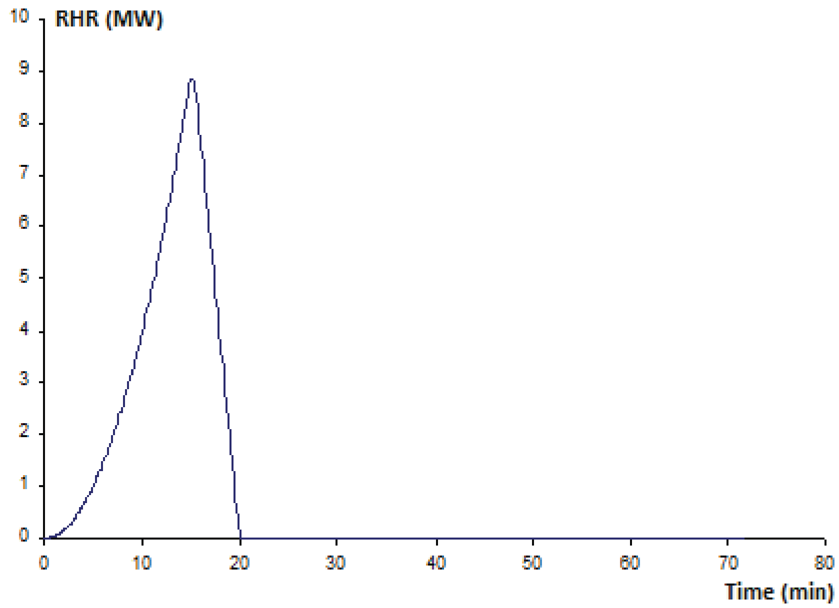


Fig. 17.4 Rate of heat release of wooden pallets fire

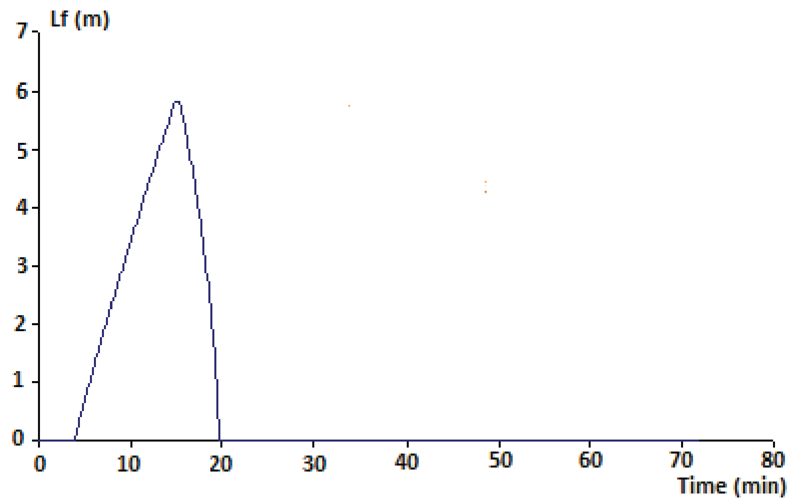


Fig. 17.5 Length of the flame of wooden pallets fire

### 17.3.2 Localised fire of high-lift truck

Rate of heat release of high-lift truck is taken as the value of fire of a car from project DIFISEK. The fire diameter D is 3.91 m. Fig. 17.6 shows time dependence of rate of heat release. The maximum heat release of the truck is 8300 kW. The fire takes 70 min. The maximum length of the flame is 4.68 m (the flame is not impacting the ceiling), see Fig. 17.7.

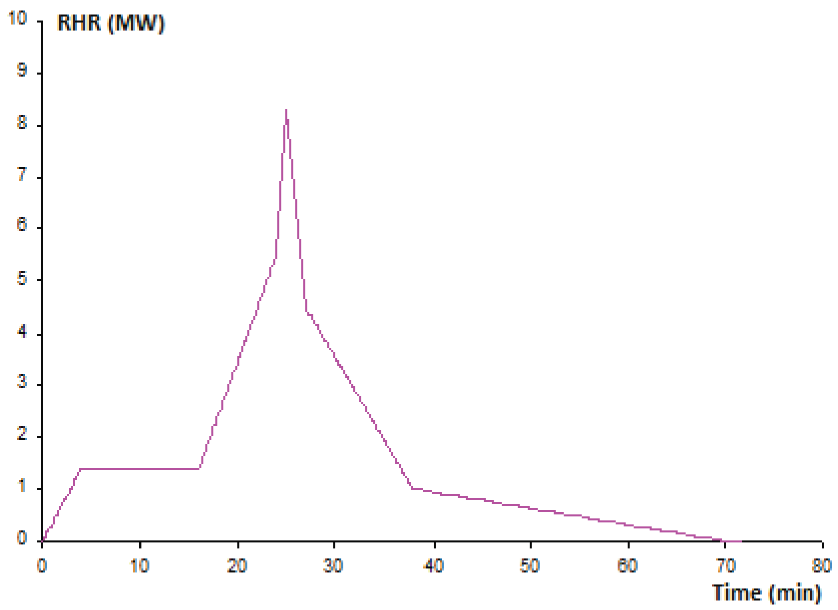


Fig. 17.6 Rate of heat release of high-lift truck fire

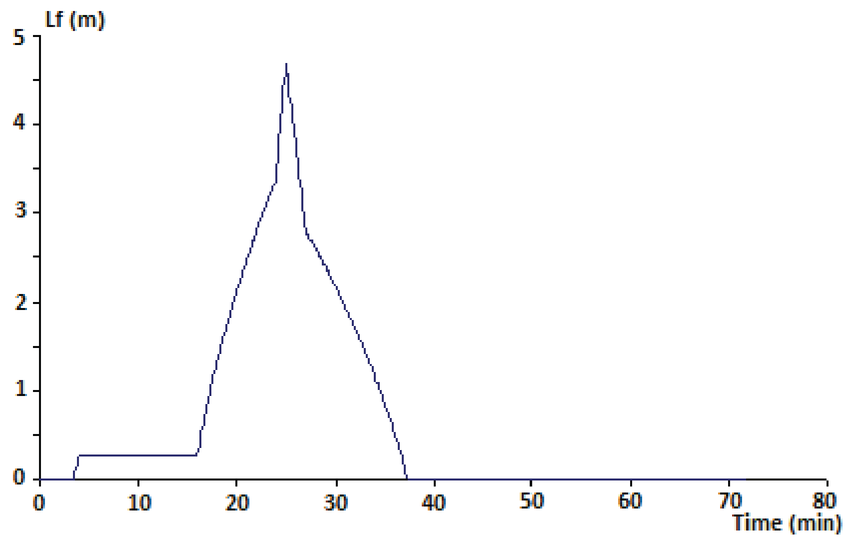


Fig. 17.7 Length of the flame of high-lift truck fire

## 17.4 HEAT TRANSFER ANALYSIS

### 17.4.1 Steel members of roof construction

The gas temperature in height of z = 10 m (height of lower chord of steel truss) is calculated using EN 1991-1-2:2002 Annex C. The maximum gas temperature during fire of wooden pallets is 221°C from 14 min 52s to 15 min 20 s.  During the fire of high-lift truck the maximum gas temperature is 184°C in 25 min, see Fig. 17.8.

The temperature of unprotected steel structure is calculated using step-by-step method from EN 1993-1-2:2005. Fig. 17.9 shows the temperature of lower chord of steel truss P1 (cross section HE180A, z = 10 m, A/V = 234 $m^{-1}$, $k_{sh}$ = 0,62) during considered fire of wooden pallets. The maximum temperature of the member is 122°C in 18 min. Temperature in time 15 min is 102°C. During fire of high-lift truck the maximum temperature of the member is 103 °C in 30 min, in 15 min the temperature is 49 °C, see Fig. 17.10. Temperatures of other members of roof structure are described in original report.

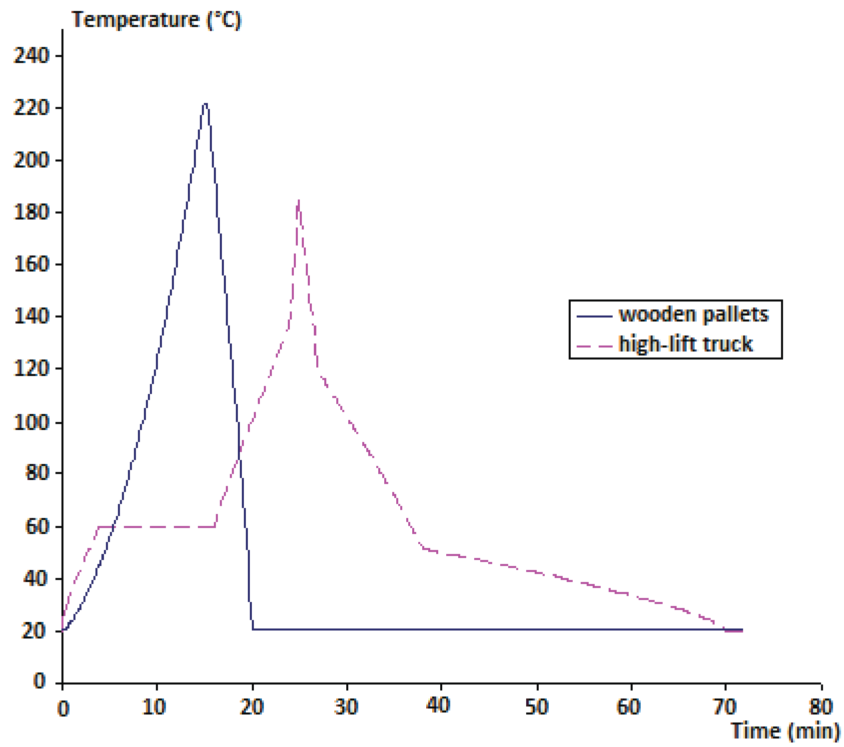


Fig. 17.8 Gas temperature in height of lower chord of steel truss

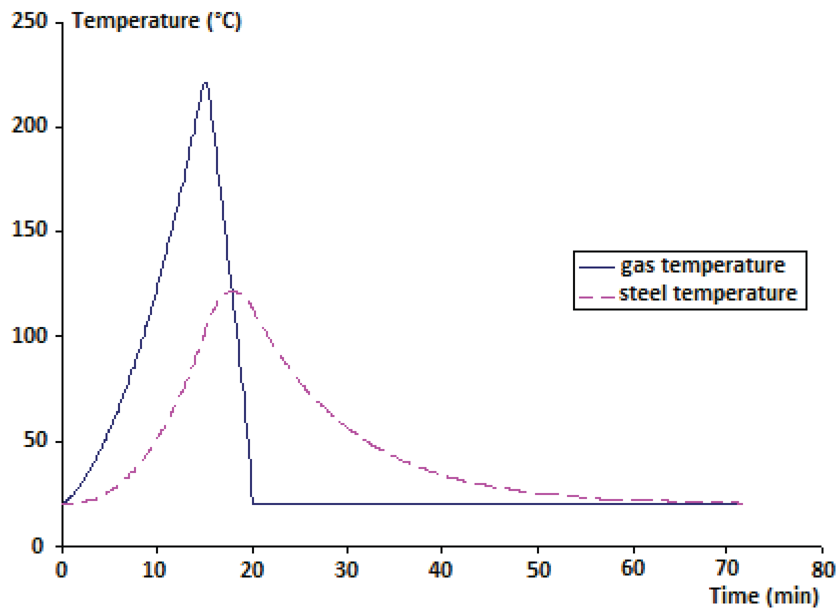


Fig. 17.9 Temperature of lower chord of steel truss P1 during fire of wooden pallets

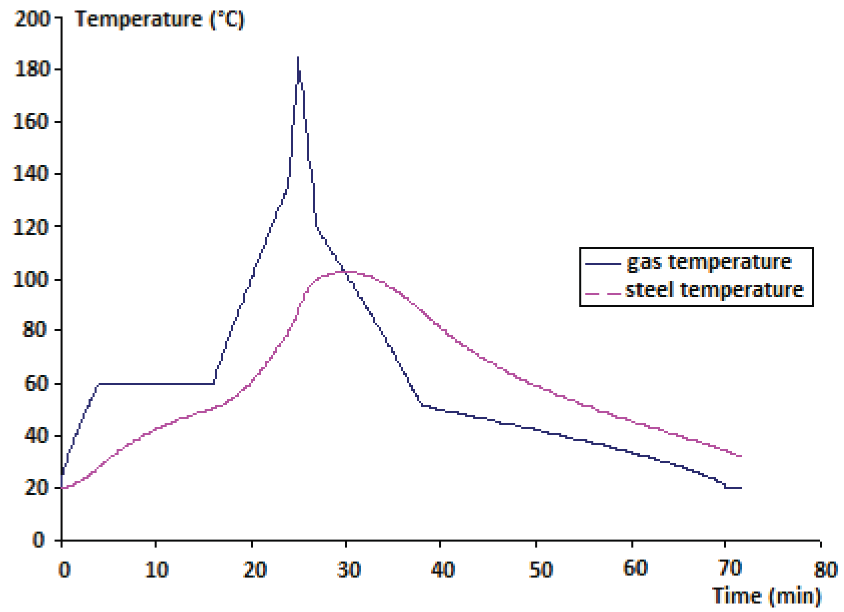


Fig. 17.10 Temperature of lower chord of steel truss P1 during fire of high-lift truck

### 17.4.2 Bearing members of cladding

The temperature of intermediate columns of the cladding during localised fire is derived using physics fundamentals and methods described in EN 1991-1-2 and EN 1993-1-2. Following assumptions are considered:

1. Flames of localised fire are substituted by cylindrical surface with diameter of the fire.
2. Columns of the cladding are not inside the cylinder. Heat transfer is realized due to radiation.
3. Temperature of column differs with its height. Heat conduction is neglected (assumption gives higher temperatures).
4. Non-uniform distribution of temperature is neglected.

Fig. 17.11 shows placing of localised fire and intermediate column of the cladding. Heat transfer due to radiation from cylindrical surface is calculated where temperature of radiant cylindrical surface depends on the height of flames $z$. The formula to calculate temperature of localised fire is used.

Due to temperature dependence of flames on the height cylindrical surface is divided into several rings with constant temperature. Temperature of selected column is calculated as sum of heat fluxes of all rings of cylindrical surface.

Surface of the column is affected by radiation from 3 sides. Using rules from EN 1991-1-2 Annex G a rectangular envelope is drawn around the cross-section of the member to describe shadow effects. In this case the configuration factor is defined numerically. The cylindrical surface is divided into rings with constant temperature. From each ring only a part which is directly visible from surface of the column is taken into account, see Fig. 17.11. The configuration factor is determined for flange and both sides of rectangular envelope parallel to column web. Final heat flux is defined as difference of heat profits and loss. The temperature in any point of the column is calculated by the aid of step-by-step method.

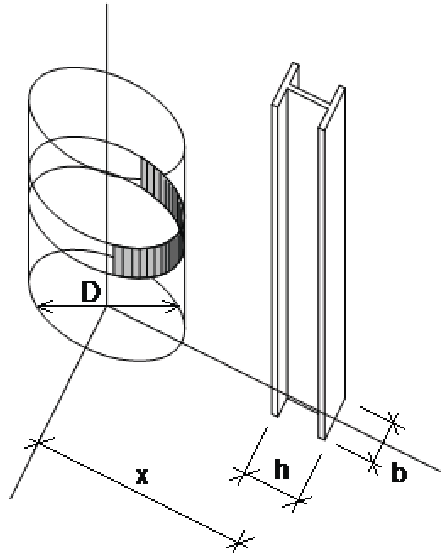


Fig. 17.11 Model of localised fire affecting intermediate column of the cladding

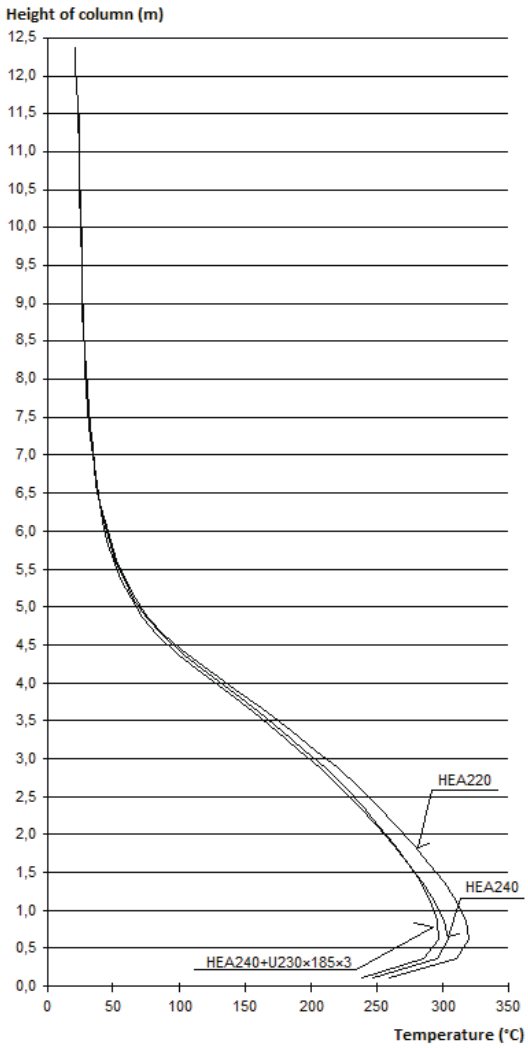


Fig. 17.12 Temperature-height dependence of column in 15 min

To calculate temperature of steel columns of cladding the fire of wooden pallets is used (the fire of high-lift truck gives lower temperature). Distance between the centre of the column and centre of the localised fire x = 1900 mm. Then face side of the column is from 285 to 295 mm from flames (depends on cross-section of the column). Columns of the cladding are of HE 220A, HE 240A, HE 240A+U230x185x3 cross-sections. Its height is 12.5 m. At Fig. 17.12 there is temperature-height dependence during the fire of wooden pallets.

Temperature of horizontal beam above the portal door is defined assuming that the fire of wooden pallets is situated under the member. Because of absence of pallets in this place the assumption is over-conservative. Temperatures of other members of cladding from hollow- square and rectangular sections are described in original report.

## 17.5 MECHANICAL RESPONSE OF THE STRUCTURE

### 17.5.1 Steel members of roof construction

Temperature in 15 min of members of roof construction does not exceed 140 °C. Reduction factor for effective yield strength $k_{y,\theta} = 1{,}00$. In this case resistance of the steel members is not influenced by fire. Considering lower load during the fire in comparison with load for limit state at normal temperature, the resistance of the members is satisfactory without next calculation.

### 17.5.2 Intermediate columns of cladding

The maximum temperature in 15 min of columns is 320 °C. Reduction factor for effective yield strength $k_{y,\theta} = 1{,}00$ (resistance of the steel members is not influenced by fire). Because of lower value of slope of the linear elastic range $E_a$ the design buckling resistance is also lower. The column HE 240A is loaded by combination of compression (weight of the cladding) and bending (wind). Its length is 12.5 m, it is held because of buckling and lateral-torsional buckling by the aid of sandwich panels.

Normal force in column is $N_{E,k} = 8{,}50\,kN$ (load acts on the width of 5.625 m, sandwich panels Kingspan with PUR insulation of 70 mm, KS 1000 TF –M/MB, 11.44 kg/m$^2$ , 13.2 m high).

Bending moment is $M_{E,k} = 58{,}6\,kNm$ (wind speed is 26 m/s, $c_e$= 1,8 (III. category), $c_{pe}$= +0.7, $w_k = 3{,}00\,kN/m'$ ).

Load combination with normal force as permanent load ($\gamma_G = 1{,}00$) and wind load as variable load ($\gamma_Q = 1{,}00$, $\psi_1 = 0{,}2$) is used.

Resistance of the combination of compression and bending of the column during the fire can be calculated with following values:

slenderness at ambient temperature is $\lambda_y = 123{,}8$

non-dimensional slenderness for the temperature $\theta$ is $\bar{\lambda}_{y,\theta} = 1{,}479$

the ratio of imperfection for buckling curves is $\alpha = 0{,}65$

$\phi_0 = 4{,}149$

geometrical imperfection is $\chi_{y,fi} = 0{,}125$

Calculated column HE 240A uses up to 10 % of its resistance during the fire of wooden pallets. Regarding the same mechanical load of other columns of the cladding next calculation is not necessary.

**17.6 SUMMARY**

The chapter summarises the examination of fire resistance of steel structure of production and storage hall object. The fire resistance of steel trusses, stiffeners and other members of roof deck and vertical cladding are investigated. Regarding large dimensions of the hall as one compartment localised fires of wooden pallets and high-lift truck are considered. The requirement for the fire resistance of the bearing structure is REI 15. According to previous calculation all members of the steel structure satisfy the resistance requirement of 15 min without any fire protection materials. The project of examination of fire resistance of steel structure of production and storage hall object in Mnichovo Hradiště was approved without any problems by the stakeholder and authorities.

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# 18 REDUCING THE RISK OF TIMBER FIRES - A CASE STUDY

Summary

A case study of a 'Serious fire' in a Timber industry located in Northern Greece is presented. In this work summarized : ignition source, fire location, first and secondary ignited materials, fire behaviour as extracted from fire investigation report. Details from fire structure report have been given (number of occupants, product/equipment, total injuries or fatalities, economical losses information and property value at risk from the fire condition etc).

Also, it is described how existed fire safety measures behave during pre - flashover conditions. Methods to improve industry fire protection have been proposed in order to avoid spreading of fire and to minimize toxic emissions. This proposition has been validated by small and medium scale experimental work.

Fire Brigade intervention and the time taken to undertake its activities at a fire scene has been evaluated.

## 18.1 INTRODUCTION

This study examines a case of a 'Serious fire' in a Timber industry located in the industrial area of Thessaloniki. This Timber industry was complied with fire safety measures as predicted by the Greek Government Desicion (1589/104/2006) "Industrial Fire protection". (so, it has been supplied with passive protection measures i.e means of escape, emergency lighting and signs , and active measures i.e. fire detection , permanent fire water supply network but with no sprinkler installation).The Timber industry was a 9.480 $m^2$ concrete building with 120 employees. Processed materials were raw wooden pine materials and final products were furniture and other wooden constructions.

## 18.2 INCIDENT ANALYSIS

The fire has been caused by spark friction originated in the production area below wooden pallets. First ignited materials were 'unprotected' wooden pallets and secondary materials were raw pine wooden material. These factors were leading to the rapid fire growth and flash over conditions. Fire almost immediately spread from first to second ignited materials. It was not contained to the room of origin and spread beyond to the whole building. Fire Compartments were inadequate to stop the fire and fire was not been be possible to be suppressed by permanent fire fighting hose reels by industrial fire staff.

Almost the whole wooden material and electro-mechanical equipment of industry has been destroyed by the fire. Estimated property loss 1,200,000 euro. On the other hand, the reinforced concrete, columns, beams performed very well in such a severe fire due to high fire resistance of reinforced concrete (above two hours). Estimated property value saved 1,000,000 euro.

(a)  (b) 

Fig. 18. 1 Fire incident during post-flashover period (a) production area , (b) storage area.

Because of the size of the fire, a site-wide evacuation was immediately initiated. Unfortunately, eight workers sustained minor injuries including scrapes and smoke inhalation.

**18.3 EMERGENCY RESPONCE**

The initial call reporting this incident was at 14.06 hours i.e in the middle of working day at 27-07-10. Timber complex had a trained and equipped Emergency Response Team (ERT) that included 20 members. On the day of the incident; 10 trained emergency responders were immediately available. Fire duration was eight (8) hours. Firefighters from the surrounding fire stations were in 'emergency alert' providing 25 fire vehicles with 60 fire fighters deployed at the scene of fire. Fire extinguished after eight (8) hours. In an effort to sustain firefighting operations while trying to establish a continuous water supply, tank water from multiple apparatus was transferred to the fire engines.

**18.4 LESSONS LEARNED**

It is clear from the above that prevention of fire spread behind the wooden first item ignited would have a significant impact on the reduction of fire losses.

In this case where the first material ignited is wood, it is considered that ignition and fire spread could be prevented or minimized by treating the timber surfaces with suitable flame retardants. Fire data on the effects of flame retardants on wooden surfaces is not available, since the relevant market is quite recent and not particularly widespread in Greece.

### 18.5 EXPERIMENTAL INVESTIGATIONS

Therefore, in order to investigate this possibility, commonly used timbers, untreated and treated were tested and compared using small scale (Cone Calorimeter) and medium scale (Enclosed Fire Rig) equipment combined with online effluent gas analysis equipment (FTIR). The most widely types of timber used in the Greek industry(Pine), were tested with bare samples, as well as using flame retardants (Small and Medium scale).

Fig. 18.2 Pine exposed at Heat flux 35kW/m2

Fig. 18.3 Untreated crib at 350sec into the test

Analysis involved thermal behavior and toxic species analysis of the samples:

- No ignition' and lower toxic emissions compared to untreated samples were observed at 35kW/m$^2$ (small scale).
- The same behavior was observed in those cases where wooden surfaces located next to ignition source had been treated (medium scale).
- 

### 18.6 CONCLUSIONS

The main factors leading to the rapid fire growth and the fire spread to almost the whole building were:

- the lack of effective fire suppression measures close to ignition source,
- the untreated wooden first and secondary ignited materials

It is proposed that the application of intumescent flame retardants on wooden surfaces located close to ignition sources in the most probable areas for a fire to break out, could be a safe and effective approach in reducing fire loss in Timber industry.

Fig. 18.4 Flame retarded Pine exposed at heat flux 35kW/m2

Fig. 18.5 Flame retarded crib at 350sec into the test

## 18.7 SUGGESTIONS

Performing of more small- and medium – scale experiments, treated with the updated technology of the intumescent paints (different parts of wooden cribs or some other form of samples), and using various ventilation rates to achieve both establishing and documentation of the contribution of intumescent technology in fire suppression, are suggested.

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# 19 TO FIRE DESIGN OF 142 m HIGH BOILER HOUSE IN LEDVICE POWER PLANT

Summary

The building of the boiler house in Ledvice power plant is 81.6 x 88.5 m with clear height 139 m, see Fig. 19.1. The structure of unique power unit with output of 660 MW was built from 2009 to 2011. The underground part of construction is steel and concrete and above-ground structural steel with reinforced concrete floor slabs. The envelope of the building was created by light hooked sandwich surface in vertical part and as composed roof surface. The fire design is based on closed boiler technology and large open space inside the building with fire risk from oil medium in 49 locations.

Fig. 19.1 Erection of the boiler house building in power plant Ledvice with two concreted braced towers providing horizontal stiffness.

## 19.1 INTRODUCTION

The boiler house with connected two reinforced concrete towers creates production block of new source of power plant Ledvice with capacity 660 MW. The main load bearing construction forms four hollow columns with sizes 2.4 to 2.4 m with thickness of the wall 65 mm to height 51 m and the structure of the boiler itself. The thickness of the wall of the column up to 51 m height is 50 mm, see Fig. 19.2. Boiler house in terms of fire safety is building with one underground and one above-ground floor. The above-ground floor is divided in different height levels by technological footbridge.

## 19.2 ENGINEERING APPROACH

### 19.2.1 Fire safety concept

The fire safety solution, see (Bebčák 2009), covers all the asked requirements for fire hazard of industrial building. Due to atypical structural solution were asked, except of Czech National rules utilising simplified procedure, see (Reichl 2008 and 2011), also the performance based evaluation of fire resistance of the main bearing structure assuring the stability of the boiler, see (Rottschaefer 2011). The procedure was developed and the results approved at Czech Technical University in Prague, see (Sokol and Wald 2011). The building divided into eight small fire compartments, as cable, pump, and turn cocks rooms and one large with boil itself with area 6 064 m$^2$. In compartment of the boiler house are except of major fire load by coal in closed technology with its fire safety equipment installed the devices with oiling. Under devices with amount of oil 400 l were create the reservoirs. The major risk locations due to oil are summarized in Tab. 19.1.


Fig. 19.2 Erection of the main frame of the boiler house (29/10/2010).

Fig. 19.3 Finalised external structure on the building of the boiler house (13/2/2012).

Tab. 19.1 The major devices with amount of the oil

| Technology | Number | Release oil volume (l) | Total oil volume (l) |
|---|---|---|---|
| Beater mills | 8 | 450 | 3600 |
| Air preheater | 1 | 320 | 320 |
| Force draft fan | 1 | 400 | 400 |
| Beater wheel carriage | 1 | 300 | 300 |
| Submerge scrape conveyor | 1 | 120 | 120 |
| Start up valves | 2 | 300 | 600 |
| HP bypass unit | 4 | 200 | 800 |

The fire scenarios considered in evaluation of fire resistance are based on localised fire of installed machine equipment. The input data were derived from parameters of the installed equipment (diameter of the fire, area of the fire, fire load density, fire growth rate, rate of heat release). The fire scenario assumed for the localised fire is based on pool fire of hydraulic oil spilled on the floor because of failure of the oil pipes. It is assumed the initial amount of oil on the floor is supplied by leak of the oil to get steady burning during the period of required fire resistance, i.e. 45 min. In addition, burning of cables on cable trays is assumed to increase the rate of heat release. The fire is modelled based on rate of heat release given in (NUREG–1805). Model of localised fire according to EN 1991-1-2 Annex C is used to predict the gas and steel temperature.

**19.2.2 Fire load by coal**

The coal is the main fire load of boiler house even, if all the facilities are designed as closed system. Three model situation of fire of coal were taken into account, see (Bebčák 2009): the fire load by coal at level 0 m due to the accident of coal mill, at level 26 m due to conveyer belts, and at level 51 m due to coaling bunker. With automatic warning, camera systems, and the firemen's attack till 10 min it is expected free burning of coal at all three potential locations till 15 min. The required fire resistance of structures due to all installed active measures and automatic foam fire extinguishers was calculated to 30 min.

The design fire of coal mill is expected due to failure of surface structure of coal closed technology and burning all coal infilling 1000 kg. A mill is located on area of 50 m$^2$, which creates characteristic fire load density of timber equivalent 248 MJ/m$^2$ Note: According to Czech standard ČSN 73 0802: 2000 are combustible materials in checked space determined by variable and permanent fire actions, which is put out to the floor area and expressed by timber equivalent with heating value 16,5 MJ/kg. The fire load by coal on conveyer belts expect the fracture of surface structure and pour out of the material from belt. Due to automatic stop alarm it is expected to burn 500 kg of coal. The area under the longest bell is 26 m x 1,5 m, e.g. 39 m$^2$ which creates characteristic fire load density of 310 MJ/m$^2$. A coaling bunker has area 50 m$^2$, which allow of burning away of surface layer of about 2 100 kg of coal. This fire load creates characteristic fire load density of 193 MJ/m$^2$.

Due to closed system for transport of coal and the active fire measurements including gas detection it is not expected potential fire of coal combustible gases.

**19.2.3 Fire load by oil**

The parameters of hydraulic oil were taken from (Iqbal and Salley 2004). Parameters of hydraulic oil were modelled like more described parameters of transformer oil. Effective calorific value of transformer oil was expected as $H$ = 46 400 MJ/kg, oil density as $\rho$ = 760 kg/m$^3$, burning mass rate as $m$ = 0,039 kg/m$^2$ and empirical constant – $k\beta$ = 0.7 m$^{-1}$. The diameter $D$ of the fire is derived from the equivalent area concept, i.e. the non-circular pool of oil is replaced by circular area of the same area.

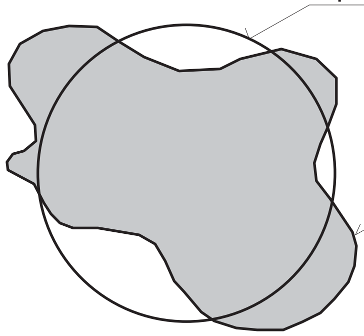


Fig. 19.4 Equivalent area of the fire

The rate of heat release of the liquid pool fire is given by

$$Q = m\,H\,A_f\left(1 - e^{-k\beta D}\right) \tag{1}$$

As the fire spreads quickly on the pool area after the ignition, the fire spreads at 10 cm/sec to 2m/sec, it is considered as simplification the rate of heat release is constant during the required fire resistance.
Duration of the fire can be evaluated according to

$$t = \frac{4\,V\,\rho}{\pi\,D^2\,m} \tag{2}$$

When necessary, the oil spill is supplied with additional oil to obtain duration of fire at least 15 min, i.e. the required fire resistance to get the maximum temperature of steel structure. The amount of oil supplied must be equal at least the amount of oil burned (the initial oil spill and increase of pool area is neglected). The amount of oil burned is given by $60mA_f\ \ (\mathrm{kg/min})$ which is $60mA_f/\rho$ (l/min).

There are cables near which can be affected by the localised fire. The rate of heat release of the cable tray was evaluated from (NUREG–1805) and added to rate of heat release from the oil spill. The rate of heat release is given by

$$Q = 0.45 Q_b A_f \tag{3}$$

where $Q_b$ is rate of heat release obtained from bench tests and $A_f$ is area of the cable tray affected by the fire. As the type of cables are not known, the PE/PVC type cables were used with $Q_b$ = 589 kW/m$^2$. This is the most flammable cable type given in the references. It is assumed the rate of heat release from the cables is constant during the fire duration as the cables are ignited from the burning oil.

### 19.2.4 Temperature of steel structure

For columns in large and high compartments, the temperature of the upper part is low and the temperature of the lower part close to the heat source would be critical. The column temperature in the lower part can be calculated when the effect of heat gained from radiation and heat loses by convection and radiation to the surrounding space is taken into account. Standard step-by-step method as in EN 1993-1-2: 2005 was utilised, for the procedure see in (Sokol, 2011). The beams were fire unprotected and fire protected. The temperatures were evaluated based on EN 1993-1-2: 2005 procedure for localised fires and the transfer of heat by step by step procedure according to EN 1991-1-2: 2002.

### 19.3 EXAMPLE OF TEMPERATURE EVALUATION - THE COLUMN CLOSE TO THE BEATER WHEEL CARRIAGE

For the calculation it is assumed there is fire of oil leaking on the floor/reservoir. The oil spills on the surface creating a pool. After the fire starts, the pool is supplied with continuous oil leaking from broken tube or fault sealing in such amount the duration of fire is 15 mins to get maximum steel temperature. The total oil spill is 300 l, i.e. 0,300·760 = 228 kg. The diameter of the equivalent circular area of the fire is

$$D = \sqrt{\frac{4\,V\,\rho}{\pi\,t\,m}} = \sqrt{\frac{4\cdot 0{,}300\cdot 760}{\pi\cdot 15\cdot 60\cdot 0{,}039}} = 2{,}88\,m \qquad (4)$$

and the equivalent fire area is

$$A_f = \frac{\pi\,D^2}{4} = \frac{\pi\cdot 2.88^2}{4} = 6.51\,m^2 \qquad (5)$$

The rate of heat release of the liquid pool fire is given by

$$Q = m\,H\,A_f\left(1 - e^{\,k\beta\,D}\right) = 0.039\cdot 46\,400\cdot 6.51\cdot\left(1-\mathrm{e}^{-0.7\cdot 2.88}\right) = 10\,218\ \mathrm{kW}\ . \qquad (6)$$

The rate of heat release from cables is calculated for estimated fire area of the cable trays equal to the total fire area $A_f$.

$Q=0.45\cdot Q_b\cdot A_f$=0.45·589·6.51=1727 kW. (7)

The total rate of heat release (oil and cables) is

$Q$=10218+1727=11945 kW. (8)

The length of the flame, see Fig. 19.5, is

$L_f$=-1:,02$D$+0:,0148$Q^{2/5}$=-1.02·2.88·+0.148·11945000$^{2/5}$=7.09 m. (9)

The virtual origin $z_0$ is

$z_0$= -1,02·$D$+0,00524·$Q^{2/5}$=-1.02·2.88+0.00524·11945000$^{2/5}$= 0.612 m. (10)

and the temperature along the height of the flame is

$\theta_g$=20+0.25$Q_c^{2/5}$($z$-$z_0$)$^{-5/3}$=20+0.25·(0.8·11945000)$^{2/5}$($z$-0.612)$^{-5/3}$=20+154.9·($z$-0.612)$^{-5/3}$. (11)

but limited to 900°C.

The column centre is located at distance 2.4 m from the centre of the localised fire. If the distance from the fire to the column increases the column temperature would be smaller. The maximum column temperature at time 15 mins is reached 2.6 m above the floor, the temperature is 169°C, see Fig. 19.6. The calculation indicates the column temperature does not exceed 350°C even during 45 mins of fire exposure if the fire would continue to burn at the same rate.

## 19.4 SUMMARY

The contribution summarises the approach for evaluation of temperature of steel structure of the boiler house in the power plant Ledvice, which supports the Fire safety solution of the building.

Acknowledgement

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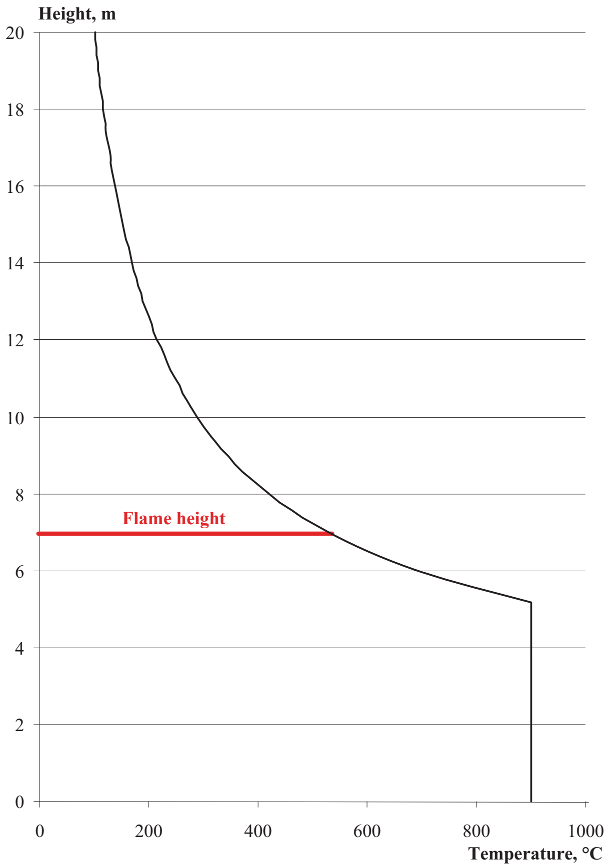


Fig. 19.5 Temperature along the flames, fire of beater wheel carriage

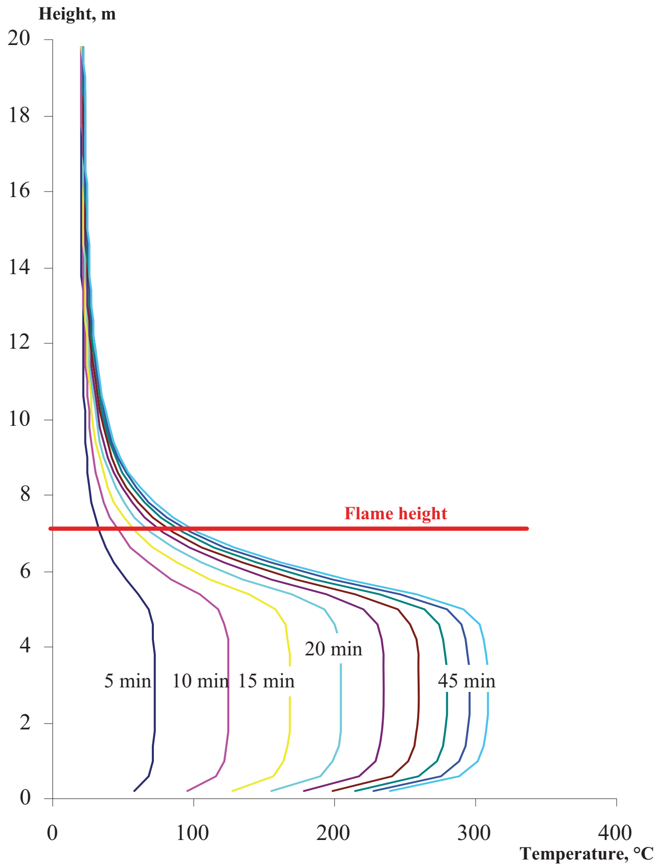


Fig. 19.6 Temperature of column RHS 1000×20, fire of beater wheel carriage

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# 20 A PRACTICAL APPROACH TO STUDY OF FIRE RESISTANCE OF A STEEL STRUCTURE WITH OPEN BUILT-UP MEMBERS AND COLUMNS

Summary

The case study exposes a practical evaluation of fire resistance of an old structure of Spanish industrial building composed of steel built-up members; the truss members are angles connected through packing plates and the columns are battened chords.

A simple calculation model was used element by element. First, heat is transferred to individual steel elements by convection and radiation in thermal study. The contributions of these two modes of heat transfer were treated by a practical approach. In mechanical study, the second order analysis was used with global imperfections. Finally, the fire resistance was evaluated R15 after some proposals.

## 20.1 PRECEDENTS OF THE CASE

The analyzed industrial building is a steel workshop that was erected more than 25 years ago. In the past, there were no specific mandatory rules in Spain to regulate this kind of industrial structures against fire, consequently the fire resistance was not taken into account in the structural design process.

In 2006, in order to renovate the activities permissions, the authorities requested the company to be in accordance with the current Spanish standard for industrial buildings (RSIEI, 2005). Following this standard, the requirement was to resist 15 minutes in an ISO 834 standard, R15. The Spanish standard (RSIEI, 2005) allows the use of Eurocode 3 (EN 1993-1-2, 2005) for checking structural fire resistance.

The company contacted the university in order to know if the current structure could be able to resist R15 without any specific fire protection.

## 20.2 BUILDING DESCRIPTION

The structure of this industrial building is made by several frames separated 5m between them. Each frame has 17,2m of span and the total length and total surface of the building are 76,2m and 1341,5m$^2$ respectively. The frame is symmetric about its centre. Fig. 20.1 below, shows a detail of geometry (Tekla, 2011).

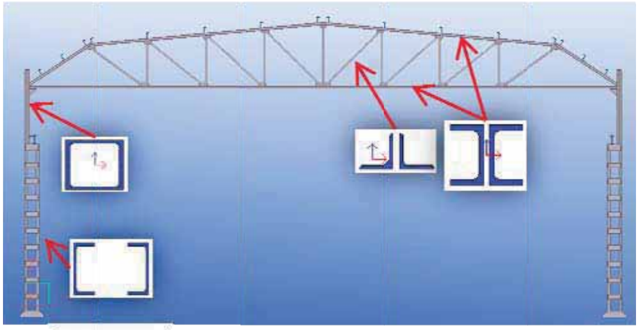

Fig. 20.1 Built-up members of the frame

Columns consist of battened built-up members of 7m height using two UPN 160 profiles, being separated at the part bellow the crane runway beam, and being a closed box above it. The column is fixed to the foundations and pinned to the truss. The upper and bottom chords of the truss are made by closely spaced built-up members using two UPN 80 profiles. They are connected through packing plates. The web members of the truss are spaced built-up members made up of battened equal angles L 40.4 or L 50.5.

Along the longitudinal direction, a resistant wall of 2,4m height offers a protection from the wind forces up to this height. As shown in Fig. 20.2, the longitudinal bracing systems in the walls and the roof provide fixed points to the frame; columns and truss. This construction results in a very light structure of 0,23 kN/m$^2$.

Fig. 20.2 Conventional bracing systems in the walls and rafter

**20.3 ASSUMPTIONS FOR THE ANALYSIS AND REGULATORY REQUIREMENTS**

The simple calculation model member by member was used to proceed in the resistance domain (EN 1993-1-2, 2005). This method is based on some appropriate hypotheses for their application to single structural members, for instance:

- No interaction between thermal and mechanical actions. Thermal and mechanical problems are solved independently.
- Effects of axial or in-plane thermal expansions may be neglected.

First, in thermal study, the uniform fire standard ISO834 was assumed for external temperature of the truss and columns in order verify R15 requirements (Fig. 20.3)

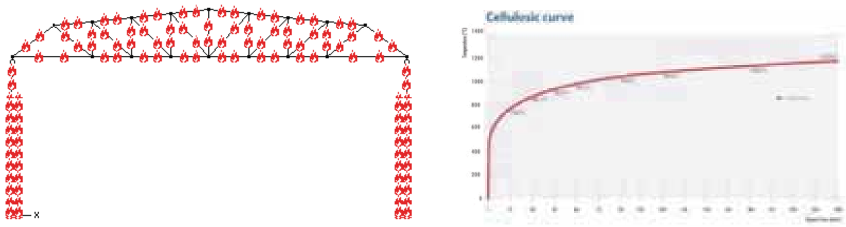


Fig. 20.3 Uniform fire standard ISO834 in the structure

Then, a global second order elastic analysis was carried out with constant elastic modulus for steel at normal temperature design, 20ºC. The boundary conditions at supports were assumed to remain unchanged throughout the fire exposure. Finally, the resistance and buckling were checked according to Eurocode 3 Part 1-2 (EN 1993-1-2, 2005). The Fig. 20.4 shows the procedure scheme.

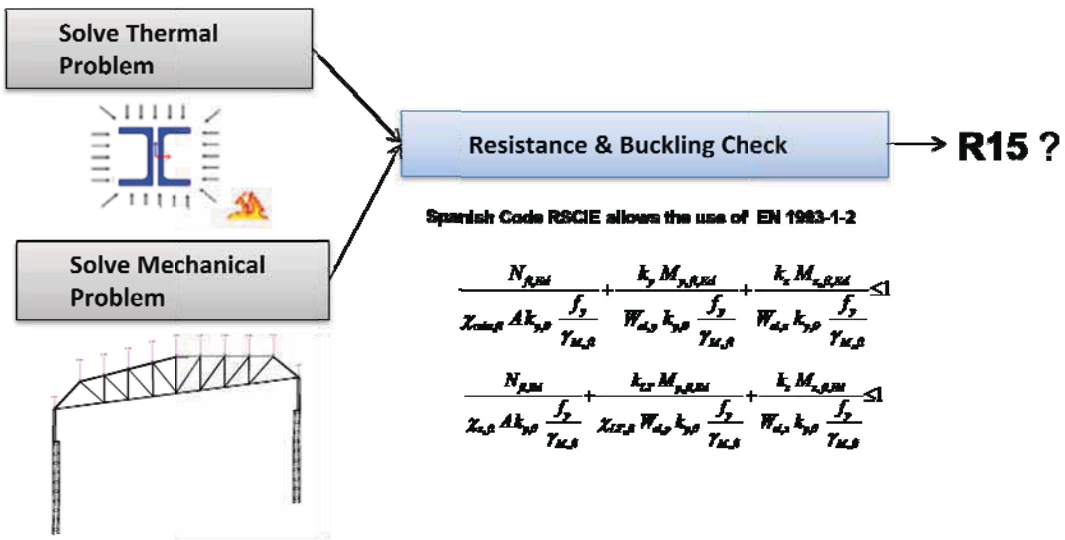


Fig. 20.4 Scheme of simple calculation model in resistance domain

## 20.4 RESOLUTION OF THE THERMAL PROBLEM

To solve the thermal problem and to know how the temperature increases in the specific built-up sections, we have used commercial finite element software. It allows the simulation of transient heat transfer in 2D free-form objects (Brista-Physibel, 2011) where the view factor is based on non-linear radiation, and the empirical convection in enclosures and boundaries. In practice, the results were very similar as use the conventional concept of the correction factor for the shadow effect for sections under nominal fire actions (EN 1993-1-2, 2005). Other models based in partial heat radiation, limited inside of built-up sections, were not considered since lower temperatures were obtained. Finally, Fig. 20.5 shows the temperature results for each cross section type after ISO 834 fire curve.

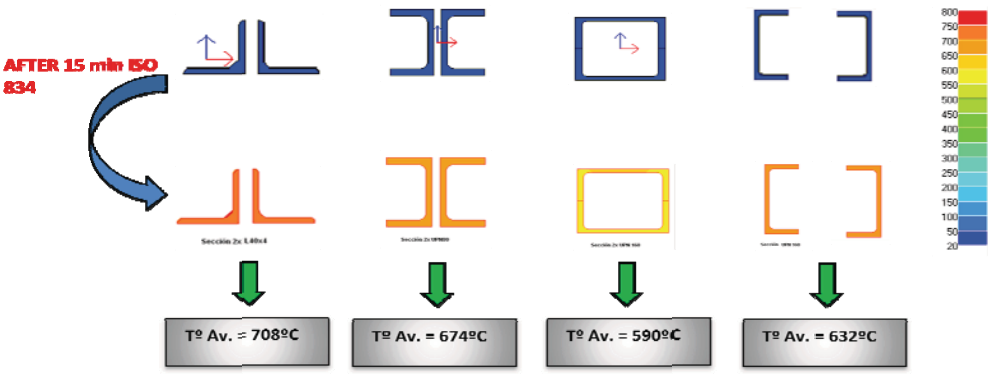


Fig. 20.5 Temperature distribution in built-up members after 15 minutes fire standard ISO 834

## 20.5 RESOLUTION OF THE MECHANICAL PROBLEM

A 2D model was used to solve global analysis and to know the internal mechanical efforts in a fire scenario, using the second order analysis by conventional structural software (PowerFrame, 2011). The frame is fixed in the longitudinal direction by the bracing systems. The next sub steps have been done:

### 20.5.1 Model

The truss beam members have modeled as their center of gravity axis lines with their respectives built-up sections in the software. In order to obtain more realistic efforts, the columns have been modeled as two different lines for each UPN chord connected by the battens in their real positions (see Fig. 20.6)

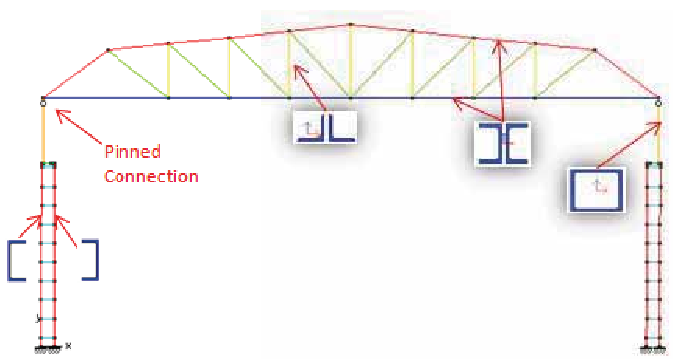


Fig. 20.6 The built-up columns were simulated by individual beams in a conventional global 2D model

Rigid links has been introduced in the model, as showed in Fig. 20.7, in order to take into account the eccentricities due to the change of section in battened columns and the exact position of the load from crane.

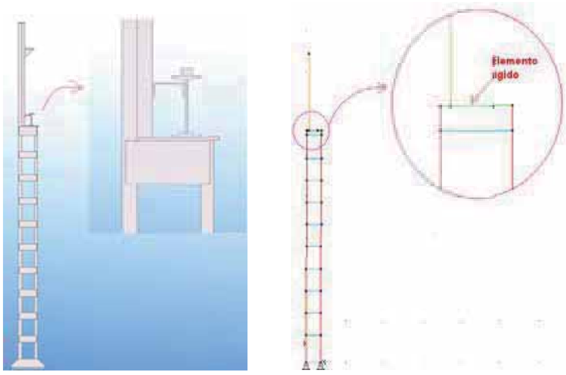

Fig. 20.7 Eccentricities of crane beam loads

### 20.5.2 Applied loads and relevant combinations

All loads taken into account are shown below. It must be said that loads for all kind of buildings have to be in accordance to the Spanish Technical Code (CTE, 2006), which describes climatic loads, majority factors and load combinations in fire scenario (see Fig. 20.8 and 20.9). Special considerations were done for the crane in fire scenario, only self weight of the crane and its accessories were used in two relevant positions (Fig. 20.10)

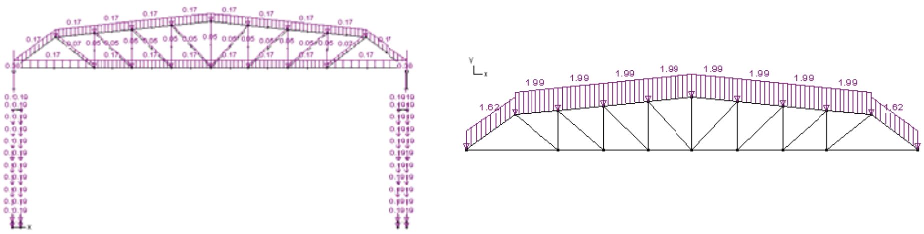


Fig. 20.8 Self weight, permanent loads and snow loads

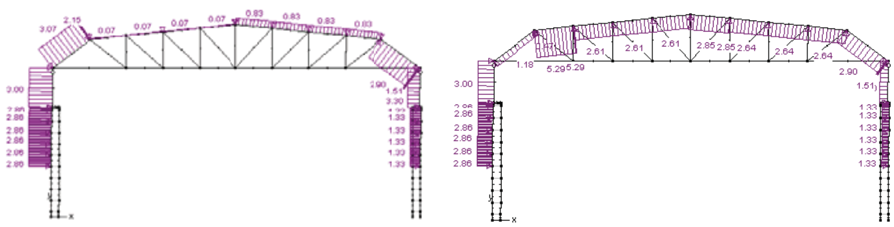

Fig. 20.9 Wind loads; two load cases from Spanish Technical Code (CTE, 2006)

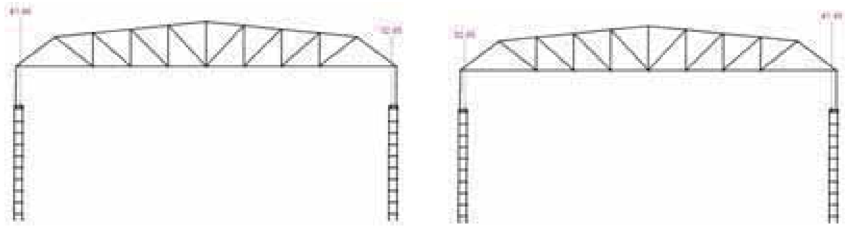

Fig. 10 Loads from the crane in fire scenario for two relevant positions

The effect of actions was determined using combinations factors $\psi_{1,1}$ and $\psi_{2,i}$ according the Spanish Technical Code (CTE, 2006):

$$\Sigma\, G_{k,j} + \psi_{1,1}\, Q_{k,1} + \Sigma\, \psi_{2,i}\, Q_{k,I} \qquad (1)$$

These values have significant differences front the recommended national values of Eurocode. For instance, in case of wind action the factor $\psi_{1,1}$= 0,5 versus recommended national value $\psi_{1,1}$= 0,2 from Eurocode.

### 20.5.3 Analysis Type

A global second order elastic analysis, including global imperfections at the top of the columns, has been carried out. No information was found about special indications to apply global imperfection in battened columns under fire conditions; so, the common value L/500 for standard conditions was used.

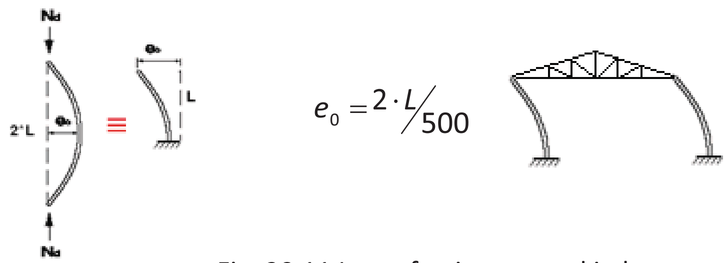


Fig. 20.11 Imperfection $e_0$ used in battened built-up columns

### 20.5.4 Flexural buckling critical length in members

The existence of two possible buckling planes shall be taken into account, requiring different checks. As long as the simplified method is used to solve the fire problem, the common buckling critical lengths used in non fire cases were implemented.

- <u>Buckling lengths in upper chord of truss</u>. The length of each split member for the buckling in the plane ($\beta_y$=1) was taken as buckling length. For the out of plane buckling, the original bracing system was modified to fix every node out plane ($\beta_z$=1). No considerations of diaphragm effect are considered.
- <u>Buckling lengths in bottom chord of truss</u>. There is no compression.

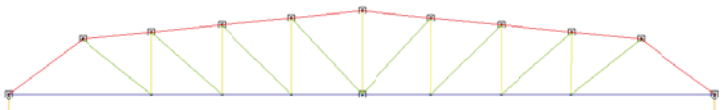

Fig. 20.12 Fixed points for out of plane flexural buckling of truss members

- Buckling lengths in battened columns. Plane buckling length of chords of columns is equal to the distance between two battens (β=1) because a second order analysis was carried out with global imperfection (see 20.5.3). In order to know the out of plane buckling length of the columns, a eigen buckling analysis was carried out using the commercial software (Consteel, 2011) (see Fig. 20.13). The second eigenvalue was used to calculate the out of plane buckling length. It was necessary a modification of original bracing system. No diaphragm effect was considered.

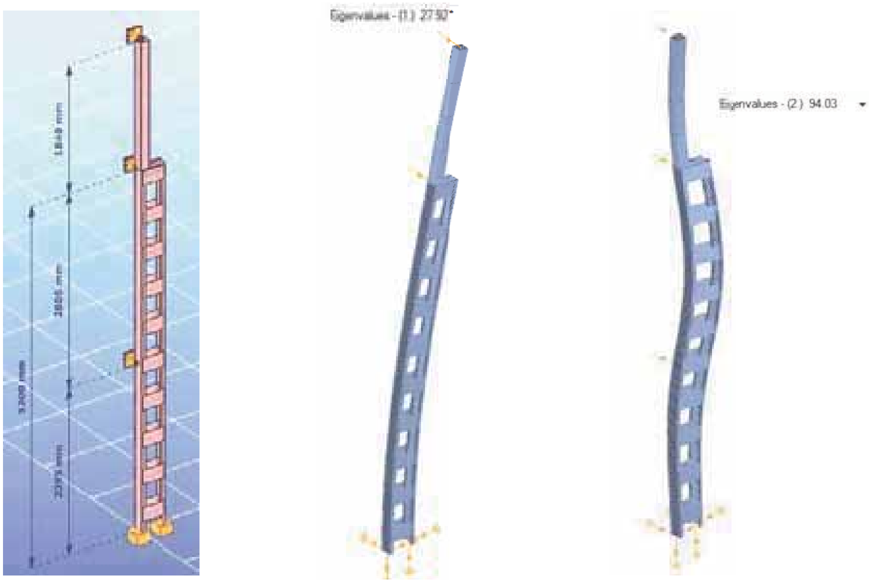


Fig. 20.13 Model with boundary restrictions and the first (in-plane) and second (out-plane) buckling modes

## 20.6 CHECKING ALL MEMBERS OF THE FRAME

For verifying standard fire resistance requirement, for instance R15, a member analysis is enough. Each member of the frame is checked at calculated temperature and compared with the efforts in fire scenario (EN 1993-1-2, 2005):

$$E_{fi,d} \leq R_{fi,d,t} \qquad (2)$$

## 20.7 CONCLUSIONS

Spanish standard for industrial buildings lets the engineer to apply modern concepts of fire engineering and allows the use of structural Eurocodes. A prescriptive requirement for structural resistance of old industrial Spanish building was verified. This building had a problematic light structure composed of steel built-up members. The thermal and mechanical study of these open members was treated by a practical approach using several commercial software. The simple calculation model in resistance domain was enough to verify the requirement of R15. Only the bracing system must be reconsidered.

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# 21 QUESTIONABLE FIRE SAFETY ASSESSMENT OF THE BAKERY PLANT BUILDING

Summary

The building broader presented in this paper is a bakery plant consisting of several premises of a different purpose and method of use, e.g.: technical facilities, production depot, distribution and storage spaces, long-term storage cool rooms, etc. The whole building that consists of single-storey technological (production and storage) part and (located on two storeys) office parts was approved as a singular fire zone with a total usable area of 6 280 m$^2$. The technological area includes production facilities, storage depots of raw materials, packages and finished products, as well as cold stores and a number of auxiliary function rooms. In the second (having two storeys)part of the building some social rooms, administrative areas and offices are localized.

The total height of the building (at the highest point) does not exceed 10.5 m. Due to the Polish regulations the parameters determining the fire-related requirements of individual structural elements of the building (especially in terms of their fire resistance) are the surface area, the average value of the fire-load density and the presence of the risk of possible explosion. The building was designed based on the assumption that the average fire-load density does not exceed the level of 1 000 MJ/m$^2$. The analysis and calculations carried out during the exploitation phase of the building confirmed the compatibilitywith the assumptions adopted, but the actual volume, estimated at the level of 974 MJ/m$^2$ proved to be very close to the limit value. Exceeding of the limit value of 1000 MJ/m$^2$ – due to provisions given in a state regulations - would automatically double the formal requirements for the resistance of the structural elements from R30 to R60. When assessing the real risk, especially in case of the large-surface-area buildings with varying ways of use of the premises, the average values of fire-load density may not properly reflect the real threat of fire. This is confirmed in the present facility, where in approximately 47% of the total area of the building the fire-load density doesn't exceed 100 MJ/m$^2$. Surfaces for which the fire load density exceeds 4000 MJ/m$^2$ (in extreme cases, it's 5644 MJ/m$^2$) represent only about 11% of the total area. It is worth mentioning that the fire-load density exceeding 4000 MJ/m$^2$ due to the national regulations and codes of design must meet the criterion of R240. A completely separate issue is the fact that the oldest part of the building was completed in violation of some basic technical and construction requirements, so that the structure of this part of the building currently does not meet any criteria for fire

resistance. This prompted the owner to implement some solutions that will not only lead the property to become fully consistent with the state regulations but also raise the level of security over the required standards, especially in the areas particularly vulnerable to fire. Presented case study shows that the method of determining the requirements for fire resistance, especially based on the average value of fire-load density, in selected cases can lead to significant underestimations and result in incorrect assessment of a building fire safety.

**21.1 GENERAL BUILDING DESCRIPTION**

The building presented in the paper is a free-standingbakery plant building, consisting of single-storey production hall with two mezzanines locatedin two separated zones of production part, and the internal patio as well as a two-storey part containing the staff rooms for employeesand office rooms.

Fig. 21.1 General view of the bakery plant building

In a production depothave beenlocalized some technological lines for bread production, storage spacesfor raw materials, packaging and finished products areas (including long-term storage cool rooms for storing bread at temperatures of about -25$^{o}$C). In addition, in this part of the building designers also predicted other auxiliary facilities, such associal rooms for employees, and both electrical or repair shops, garage offorklift trucks, and a number of technical areas housingchillers, water treatment plant, electrical switching station, boiler room and air compressor operating a pneumatic transport of flour.

The ground-floor of the two-storey office part houses reception room, security agency office and hygienic rooms as well as restroom facilities for workers. On the second floor of the building a social and administrative area is located.

Functionally plant can be divided into *technological part* and the *social-administrative office part* (Fig. 21.2).

The technological part is consisting of:

- ✓ the storage-room of raw materials,
- ✓ silos for flour (2 external and three inside the building),
- ✓ two independent production lines,
- ✓ the packaging warehouse,
- ✓ a refrigerated warehouses and related technical areas,
- ✓ the storage of finished retail products with a distribution centre,
- ✓ the washing station for container boxes.

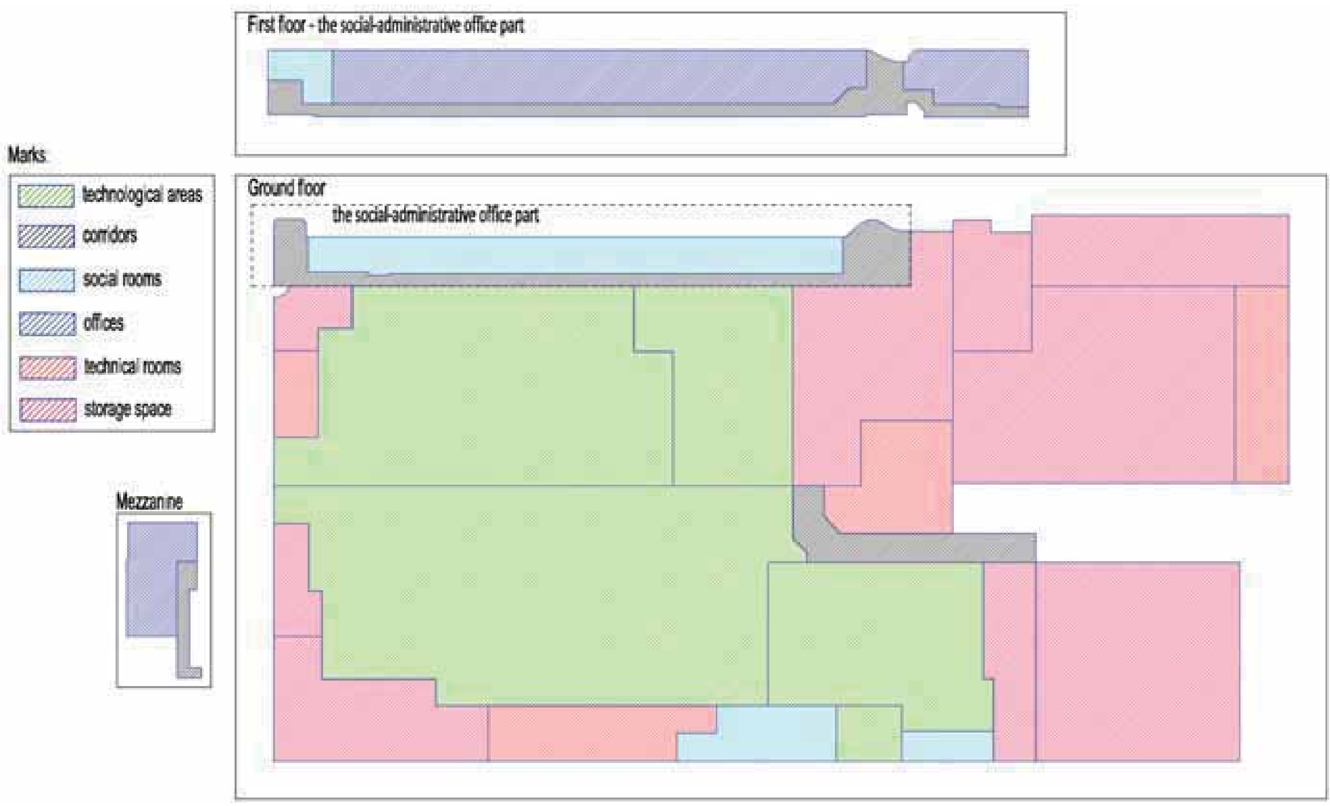


Fig. 21.2 Function of premises

The building in its present form was erected in three stages (Fig. 21.3). Initially, it was the production hall only, which was then extended to other facilities areas as the hall for the second production line and the social-administrative office part. Ultimately, in the last stage - the warehouse for retail products and a new washing station for container boxes were built.

The building was erected as of the steel structure object. Curtain walls and internal partitionshave beenmade of sandwich panels and the roof is finished with trapezoidalmetal sheets with appropriate thermal insulation layers. The social-administrative office part uses mixed steel and reinforced concrete structure, supported at the one end on the main structure of the technological (production) hall. Curtain walls have been designed as a multi-layered structure–built with clay brick tiles, insulated with polystyrene boards and finished with clinker. Partition walls are made of brick or plaster-cardboards. Roofing is finished with sandwich panels with trapezoid sheets located on both sides. The wall located between the office and technological partshave been made of sandwich panels.

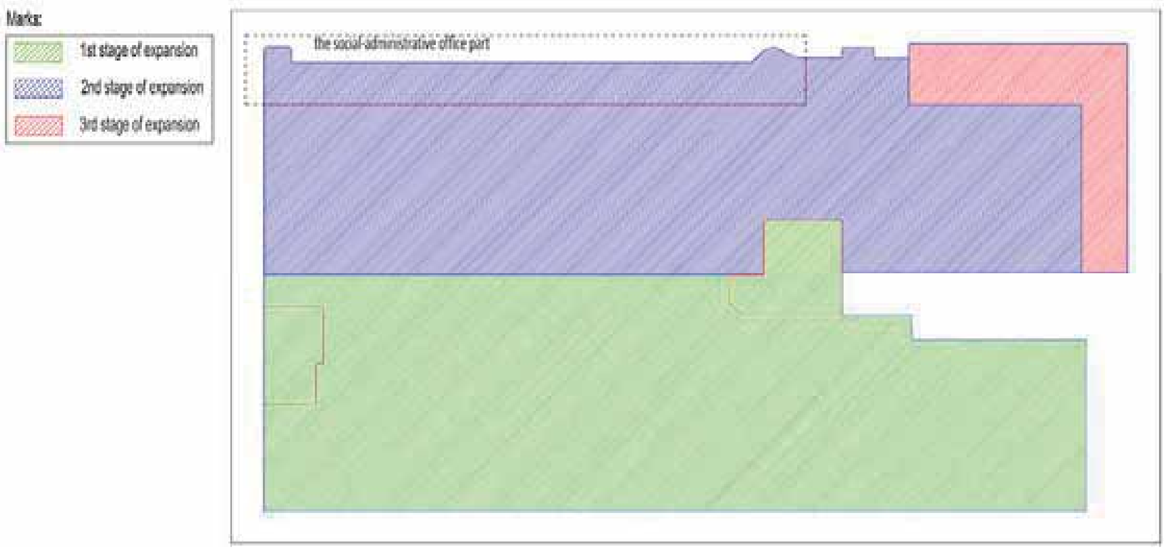


Fig. 21.3. Stages of expansion

The number of employees of the plant fluctuates around 189, while at the same time about 65 people may stay in the building.

The property now forms a vertical projection of the rectangle-like shape with external dimensions of 127m x 67m. As it was mentioned before building height does not exceed 10.5 m and the usable area is equal 6 280m$^2$.

**21.2 FIRE SAFETY ENGINEERING SOLUTIONS USED**

According to available documentation, despite the fact the building consists of premises of different functions, the object has been designed to function entirely as a wholein which it was assumed that:

- the fire load density does not exceed 1000 MJ/m$^2$, and
- does not contain any spaces and hazardous areas.

Due to Polish regulations and considering assumptions given above,it is allowed to assess the building as a singular fire zone. In the newest part of the property (storage and distribution of retail products area and washing station for container boxes) the structure was protected using the intumescent painting system to ensure designed flame resistance level.

The building is equipped with a fire plumbing system, which in the social-administrative office area and in the technological area located in the oldest part of the object, is finished with hydrants DN25. In the remainingparts of the building the fire hydrants type DN52 have been installed. The property is also equipped with an electric current fire breaker.

Important from the viewpoint of evacuation safety is the fact that the maintenance of refrigeration equipment in the facility uses ammonia system. For this reason, in the room where the chillersare located have been installed asystem of ammonia leak detection and signalling, as well as the mechanical ventilation. Some escape masks with ammonia absorbent filters were deployed in the building to ensure the emergency escape in case of ammonia system's failure.

The emergency lighting lamps have been positioned along the emergency exit to facilitate the exit of the building by employees in case of emergency.

Evacuation of the building was planned by a number of emergency exits leading directly out of the technologicalpart of the propertyand through the two staircases in the social-administrative office area. In the walls between the staircases and corridors the fire doors of EI30 fire resistance were mounted. The corridor wasprotected with a smoke insulating door mounted in a halfway of its length.

In addition, in a close neighbourhood of the building four external fire hydrants were placed (powered by deep well which is the main source of water for technological purposes and the living), which provide a security source of water to extinguish the fire for the fire service.

### 21.3 REGULATORY REQUIREMENTS

Due to the Polish state legislative regulations the classification of buildings for fire safety is based on the *total height* and *the way, the building is used*.The buildingsin general are divided into five groups: *low* (no higher than 12m), *medium-high* (more than 12m to 25m), *high* (more than 25m to 55m) and *high-rise* buildings (over 55m).

After determining the *primary way of use* or *function of the building*, one can assign it to one of three categories: *risky to humans* (objects where the primary function is associated with at least temporary stay of people), *manufacturing - warehouse* (where the primary function is to produce and/or storage) and *livestock* (designed for plant growing and/or animal breeding).

Due to presented rules the object in question, described in the paper, should be classified as the"*low*" building (below 12m of height). Uncertainty may arise in assessing the primary function and qualifying it dependently on the way of usage. There are no special doubts about the fact, that part of the building serves as the technological production and warehouse zone. The question arises how about the social-administrative office area? This is the area which primary function is associated with its occupation by people for more than 4 hours/day. In such cases, the technical state regulations require the separation of different parts of building with various functions as a separate fire zones, unless they are linked functionally.

Except the two previously mentioned parameters there are two other ones that have a direct impact on fire protection requirements for production/storage buildings: the *presence of the risk of possible explosion* and the *average value of fire-load density*.

As mentioned earlier, the object wasdesigned and built to function entirely as a whole with the fire-load density less than 1000 MJ/m$^2$which do not contain any hazardous areas or spaces. In terms of height the building has been classified as a *"low"* category , due to way of use as the production/warehouse industrial object.

These are relatively comfortable assumptions, which allow the design and execution of the object as a singular fire zone (with no separate fire zoning for office and technological areas), while putting little demands in terms of fire resistance for the main construction of which will be discussed later.

Having information about a classification of a building, based on its height and the density of fire-load one can determine the so-called *"class of fire resistance"* (denoted by one of the five successive letters of the alphabet: A, which is the highest to E, which is the lowest). Assignments are made based on Tab. 21.1, below.

Tab. 21.1. The required global fire resistance class of a building

| Maximum fire-load density in a fire zone of the building Q [MJ/m$^2$] | Single-storey building (without limitations of height) | Multi-storey building | | | |
|---|---|---|---|---|---|
| | | **low (L)** | Medium-high (MH) | High (H) | High-rise (HR) |
| *1* | *2* | *3* | *4* | *5* | *6* |
| Q =< 500 | "E" | "D" | "C" | "B" | "B" |
| **500 < Q =< 1000** | "D" | **"D"** | "C" | "B" | "B" |
| 1000 < Q =< 2000 | "C" | "C" | "C" | "B" | "B" |
| 2000 < Q =< 4000 | "B" | "B" | "B" | * | * |
| Q > 4000 | "A" | "A" | "A" | * | * |

The legislation contains a number of cases in which the required class of fire resistance can be lowered or when it should be increased. Directly from a determined class of the global fire resistance precise requirements for fire resistance of individual building components can be derived, as shown in Tab. 21.2, below.

Tab. 21.2. Requirements for the majorstructural elements regarding criteria for theglobal fire resistance class

| **Specified Fire Resistant Class of a Building** | **Fire Protection Requirements for Major Structural Elements of Buildings (rates in minutes)** | | | | | |
|---|---|---|---|---|---|---|
| | Main supporting structural members (columns, walls) | Structure of the roof | Floor slab | External wall | Internal wall | Roof finishing layers |
| "A" | R 240 | R 30 | REI 120 | EI 120 (o-i) | EI 60 | R E 30 |
| "B" | R 120 | R 30 | REI 60 | EI 60 (o-i) | EI 30 | R E 30 |
| "C" | R 60 | R 15 | REI 60 | EI 30 (o-i) | EI 15 | R E 15 |

| | | | | | | |
|---|---|---|---|---|---|---|
| **"D"** | **R 30** | **(-)** | **REI 30** | **EI 30 (o-i)** | **(-)** | **(-)** |
| "E" | (-) | (-) | (-) | (-) | (-) | (-) |

It should be noted that in this specific case no additional solutions that make it possible to reduce the required fire resistance class D were applied and thus the supporting structure must meet the criterion of R30 in case of fire.

On the basis of aclassification of a building, based on its height and the value offire-load density one can also set an acceptable fire zone area in the facility. The final value is dependent not only on the mentioned parameters but also on a number of storeys and the presence of zones with risk of possible explosion. These relationships are summarized in the Tab. 21.3, below.

Tab. 21.3. Allowablesize of a fire zone

| Type of fire zone | Fire-load density Q [MJ/m$^2$] | Allowable size of a fire zone [m$^2$] | | |
|---|---|---|---|---|
| | | In a single-storey building (without any limitation of height) | **In a multi-storey building** | |
| | | | **Low & Medium-high (L) and (MH)** | High & High-rise (H) and (HR) |
| 1 | 2 | 3 | 4 | 5 |
| Fire zones with potentially explosive premises | Q > 4 000 | 1 000 | * | * |
| | 2 000 < Q ≤ 4 000 | 2 000 | * | * |
| | 1 000 < Q ≤ 2 000 | 4 000 | 1 000 | * |
| | 500 < Q ≤ 1 000 | 6 000 | 2 000 | 500 |
| | Q ≤ 500 | 8 000 | 3 000 | 1 000 |
| **Remaining fire zones** | Q > 4 000 | 2 000 | 1 000 | * |
| | 2 000 < Q ≤ 4 000 | 4 000 | 2 000 | * |
| | 1 000 < Q ≤ 2 000 | 8 000 | 4 000 | 1 000 |
| | **500 < Q ≤ 1 000** | 15 000 | **8 000** | 2 500 |
| | Q ≤ 500 | 20 000 | 10 000 | 5 000 |

As seen above, allowable area of the fire zone in this building is equal 8 000m$^2$. As with the requirements for fire resistance, the surface can be changed using the appropriate solutions in terms of fire protection systems (e.g. smoke removers or permanent fire extinguishing devices).

### 21.3.1 Requirements for the evacuation

Polish regulations impose an obligation to ensure the evacuation of any room that may be occupied by people. By this it is meant to enable a safe emergency exit to leave the building, either directly or through channels of general communication to any safe place outside the building or to an adjacent fire zone. The law differentiates between the two parameters of the length of an escape route: *transition routes* and *access routes*.

*The transition routes* apply for rooms and spaces where there are no separate corridors. The length is measured from the farthest place where the individual human being can stay, to the exit on an escape route (corridor), or to another fire zone, or outside the building. Measurement should be made by the shortest possible route. In the case concerned, length of the transition route should not exceed 75 m. One can increase the permissible length of about 25% but only in areas with a height exceeding 5 m. It should be noted that if in a building there would be a room of potentially possible explosionit's permissible length of the evacuation passage is then reduced to 40m.The evacuation route must be of adequate width to be calculated in proportion to the number of persons for whom it is designed, taking at least 0.6 m per 100 people, but not less than 0.9 m. In the object in question the latter condition applies.

*The access routes* shall be measured in the axis of an escape route starting from the exit out of the room on this route up to the exit to another fire zone, or outside the building, or the enclosed stairway. The stairway should be then closed by the door of the fire resistance class EI30 at least, and equipped additionally with devices to prevent against smoke accumulation or used to smoke removal. In case of presented building, the length should be 30m when only one access route is provided (including no more than 20m of the horizontal escape route) or 60m with two or more routes designed. In the latter case it's allowed that the length of the second access route can be about 100% greater than the shortest one in the same fire zone. These access routes cannot overlap or cross with each other. With the presence of explosion endangered areas, what seems the crucial role in case of presented building, the length of access routes should be reduced to 10m when exists only one access route or to 40m with two or more routes designed.

Moreover, due to technical and building-related regulations:

- total width of the door, which exits from the room, should be calculated in proportion to the number of people who can reside inside at the same time, taking at least 0.6 m of width for 100 people, but not less than 0.9 m,
- minimum width of staircase should not be less than 1.2 m and the width of landing not less than 1.5 m - both values are measured in the most unfavourable location and finally the total required width is determined according to the rule providing of 0.6 m for every 100 people,
- an emergency exit door of a building designed for more than 50 people should open outwards,
- width of the outside door mounted on the emergency exit of the building (unless they are not the only exit from the separate room), and the width of the door to escape through the stairway leading outside the building or to another fire zone, should not be less than the required width of a staircase (in case of the concerned building - 1.2 m).

**21.3.2 Requirements for fire equipment and other installations**

Due to the fire requirements in the concerned building, fire equipment comprises of:

- fire plumbing equipped with hydrantstype DN52 (in the production part of the building) and DN25 (in the officezone),
- fire power switch.

The building is also equipped with a gas installation. The gas is used in a gas boiler with a capacity of 340 kW and in the process of baking. According to the state technical requirements - when the total nominal output of gas appliances installed is greater than 60 kW then such areas should be equipped with signalling installation and shut-off the gas supply devices. A valve shutting-off the gas supply, which is a component of signalling installation and shut-off devices should be located outside the building, between the main valve and the entrance of gas pipeinto the building. Additionally, the building concerned also requiressome water for fire protection provided, (in the volume of 40 $dm^3/s$), and convenient access for the fire-fighting brigades, i.e. the fire road.

## 21.4 ASSESSMENT STRATEGY

The basis for the verification of previously mentioned formal technical and fire requirements has become the idea to extent a building with a small storage space for baking molds. The room was to enlarge the existing fire zone which, according to Polish regulations means that a building must meet current fire safety requirements, regardless of legal status at different stages of reconstruction. Due to the owner's wish -next tothe review of existing current solutions for compliance with fire safety rules some assessment was made what level of safety all these solutions provide. Assessment for compliance of applied solutions with current fire safety rules and evaluation of security level of the building were based on the so-called *"Terms of fire protection"* given in form of fourteen key-points defining the main requirements for building fire safety, i.e.:

1. Size of floor area, height and number of storeys,
2. The distance from neighbouring objects,
3. Fuel/thermal parameters of present flammable substances,
4. Fire load density,
5. Category of building understood as the substitute of risk to people, estimated number of persons enable on each floor and/or individual rooms,
6. Assessment of explosion risk at rooms and outdoor spaces,
7. The way the object is divided onto separate fire zones,
8. Global fire resistance class of the building, fire resistance class of building components and the degree they spread the fire,
9. Conditions of evacuation, emergency (evacuation and backup) and obstacle lighting systems,
10. The applied methods of fire protection to protect the service installations,

11. Selection of fire-fighting equipment used in the building,
12. Number and type of fire extinguishers the building is equipped with,
13. Water supply for external fire-fighting,
14. Presence and quality/parameters of fire roads

**21.5 DESCRIPTION OF THE APPROVAL PROCES**

The investment process begins with the classification of the object in terms of the total height and the way, the building is used. Then, the designer assumes according to their knowledge the presence of a potentially explosive premises and the level of fire load density (average value in a whole fire zone).

The next stage of investment process is to obtain a building permit issued by an authorized body of territorial administration based on industry experts' agreement in relation to: fire protection, safety and hygiene of work and epidemiological-sanitaryas well. After obtaining a building permit an object is being erected under the supervision of a certified construction manager and supervising inspector. Supervising persons after completion the construction work shall declare that the execution of an object was done according to the project design, and the law, and the principles of technical expertise. This statement makes these people responsible for the correct execution of the property, even in case of some design errors. If any of them occur and are found out during the erection of the building it is the duty of the supervisor to stop ongoingwork and discuss with the designer what kind of alternative solution exists which is consistent with the rules and technical expertise.

Due to the Polish legislation, existing buildings should be adapted for current requirements of fire safety, such as: development, superstructure, reconstruction and changes of function (the way, the building is used) or when based on legal regulations [3]if it's considered threatening people's lives.

The building presented in this case study, as it was already mentioned, was expanded twice during its lifetime.

State authority empowered to control buildings in the meaning of fire safetybefore they are permitted to be exploited(which in Poland is the State Fire Service), during the reception after the third stage of development pointed out several inconsistencies which should be adapted to current requirements.

Inconsistencies with the provisions stated are listed below:

- DN25 hydrants used in the oldest part of the technological hall in contrary to required DN52 type,
- corridor (in the office part of the building) of a length exceeding 50 m not divided onto the shorter segments with the smoke-tight door,
- surgical stairs in the stairwells of the office part of the building.

In the first two cases, the necessary changes were done. Implementation of the third one would require demolition of the existing stairs. Polish regulations allow for the use of alternative solutions that would compensate the non-compliance. The recommended available solutions are defined by certified fire safety expert. However, they must be approved by the State Fire Service. In the presented building it was suggested to close the stairwells with doors satisfying the criterion EI30 of fire resistance. Finally, after applying the suggested solutions the property was considered compliant with the technical rules, but some doubts about the level of safety guaranteed by protection used were indicated.

### 21.5.1 Fire load density

In the analysed building the fire load density (due to the assumptions of project) does not exceed 1 000 MJ/m$^2$. Based on detailed analysis of the distribution of a fire load density it can be assumed that the design assumption is correct (Fig. 21.4). It should be noted that the presented value is the average one. When assessing the real risk, especially in case of the large-surface-area buildings with varying ways of use of the premises, the average values of fire-load density may not properly reflect the real threat of fire. This is confirmed in the present facility, where in approximately 47% of the total area of the building the fire-load density doesn't exceed 100 MJ/m$^2$. Surfaces for which the fire load density exceeds 4000 MJ/m2 (in extreme cases, it's 5644 MJ/m$^2$) represent only about 11% of the total area. In the lowest range of the fire load density which one can find in the technical regulations (i.e. of less than 500 MJ/m$^2$) falls to approximately 74% of the floor area of the analysed object.The estimated average fire load density [6] reached 974 MJ/m$^2$. Such a low value with such a large share of the floor area with relatively small or even negligible fire loadsuggests that there exist someareas where the small area accumulated large quantities of combustible materials. In these places the fire load density reaches a value that extends beyond the other end of the scale which we met earlier in regulations - over 4000 MJ/m$^2$ (compare Table X.1). Surfaces for which the fire load density exceeds 4000 MJ/m$^2$ (in extreme cases, it's 5644 MJ/m$^2$) represent only about 11% of the total area. In these areas, construction of a building designed and executed based on the assumption quoted at the beginning of the chapter is not adequately prepared for the likely fire conditions that may occur there.

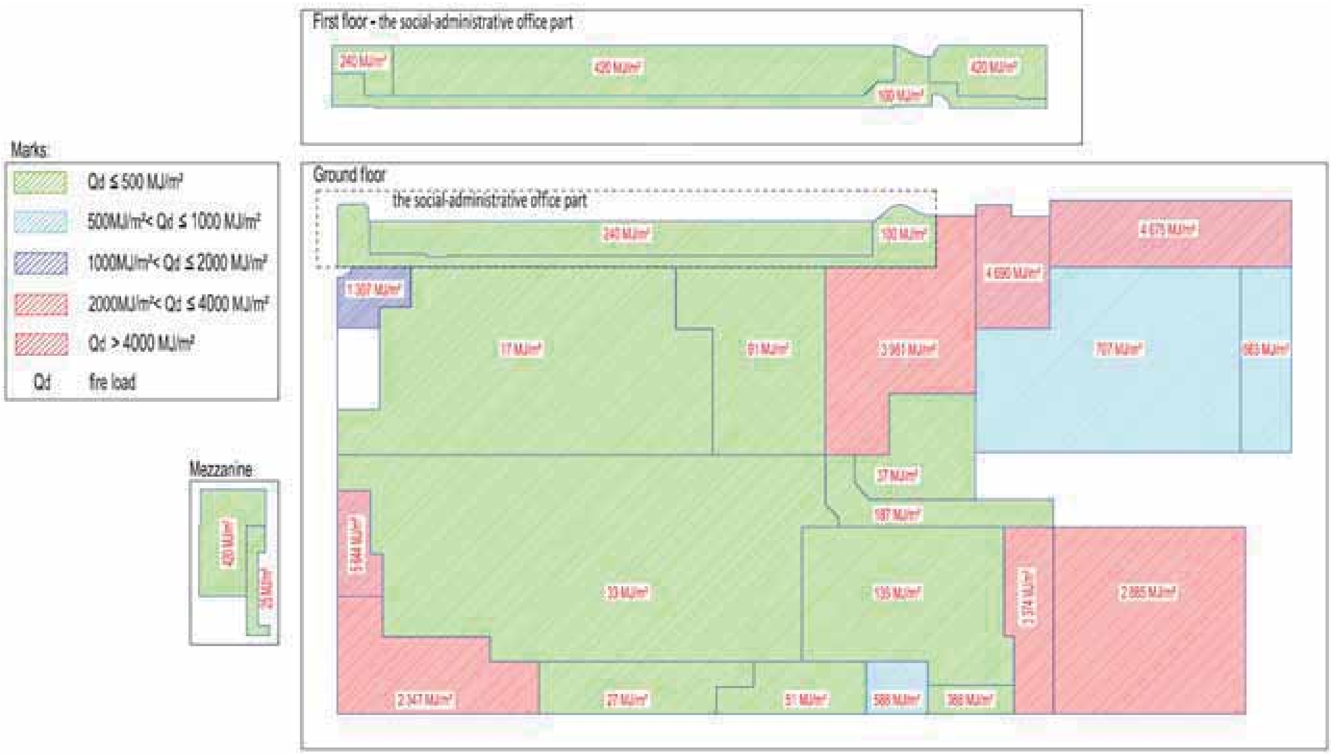


Fig. 21.4 Distribution of a fire load density

### 21.5.2 Fire zoning

Claiming that the office/administrative part of the building is linked functionally with the technological/production part (what was the main argument that made it unnecessary to divide the object onto the separate fire zones) seems rather questionable.

In addition, the allowed surface area of individual fire zone in a low multi-storey building, in which there are no potentially explosive areas, and the fire load density fits the range 500 <Q ≤ 1 000 is 8 000 m$^2$ (compare Tab. 21.3). Surface of the analysed object is now nearly 79% of the limit.

As already mentioned, the design assumptions have been positively verified with the amount of combustible materials taken for analysis on the basis of the state in May 2011, when a calculated fire load density was 97% of the assumed threshold, so very close to the limit value.

If – theoretically- the established threshold of 1000 MJ/m$^2$ would be exceeded(which in practice is possible when storing larger quantities of materials, raw materials or products) the requirements for the maximum fire zone area would increase, and the real floor area of the analysed facility would then reach 157% of the limit.

The existing building and its fire zoning is consistent with the current rules, but it cannot be regarded as beneficial in the meaning of safety reasons.

### 21.5.3 Assessment of explosion risk

Design assumptions imply that there are no rooms with potentially explosive atmospheres. A preliminary assessment confirmed this assumption. It does not mean that inside the building there is no risk of explosion. Factor posing a threat of explosion may be, for instance, the dust layer located on the floor, machines, or in inaccessible locations, which can be easily undermined, and mixed with air by a gust of wind. The inspection carried out in the building revealed that there are some surfaces permanently not cleaned, which were enveloped by the dust layer. In some cases, these were the areas of elevated temperatures: the electric motors or enclosures of bakery machines.

### 21.5.4 Fire resistance of building elements

The bakery plant was designed and built in class D of fire resistance. As mentioned before, the areas of the building where the average fire load density exceeds 4000 MJ/m$^2$ represent about 11% of the area. When the requirements would be imposed on the basis of local rather than average fire load density distribution - in these areas they would be the highest possible, (Class A of global fire resistance).

In addition, it should be also noted that in spaces executed during the first two phases of investment (the majority of the area of the plant) the main structure of on object is not protected to the required level of fire resistance equal R30.

### 21.5.5 Water for external fire-fighting

Water for external fire-fighting provide four external landline hydrants located on the property, supplied with its own water intake and one fire hydrant located in a neighbouring plant, water supply network powered. The owner of the neighbouring building, where this hydrant is localised has agreed to use it to protect the bakery plant concerned. These two fire hydrants located on the bakery site do not provide adequate amounts of water for fire-fighting purposes and do not meet proper efficiency requirements.

Analysing the technical capabilities of the water source used, which is the well with a capacity of 50 m$^3$/h it should be noted that this amount would only be sufficient to meet water demand for one outside hydrant only (13.9 dm$^3$/s at the required flow 10dm$^3$/s). Taking into consideration the fact, that the same source is also providing water for the internal water fire-plumbing system, it must be assumed that it is insufficient to power the external hydrants. Therefore it was necessary to ensure supply of water in a proper amount which results from a missing relative flow and the estimated duration of the fire, according to Polish Standard PN-B-02852:2001. In the presented case a missing amount of water is equal 30 dm$^3$/s and a relative duration of the fire is equal to 3600 seconds. So that it’s obligatory to guarantee reserve of water to extinguish a fire from the outside in the amount of 108 m$^3$.The solution that complies with the rules is the use of fire-fighting water tank complying with the requirements of Polish Standards.

## 21.6 RECOMMENDATIONS FOR THE FIRE ENGINEERED SOLUTIONS TO BE APPLIED AT THE SITE

Items below summarize some recommendations arising from the analysis carried out and the local considerations that allow to adapt the building for fire safety conditions:

1. It is reasonable to design additional protection in areas where a large fire load density occurs in tandem with the high threat of fire. The recommended solution is a permanent water or foam extinguishing equipment.
2. Some fire and explosion prevention recommendations were formed, aimed at minimizing the risk of explosion where it was indicated as a basic function the need of systematic cleaning of surfaces on which dust of flour and/or sugar can settle.
3. For fire zoning: It was recommended to separate the office part of the building from the technological part (as different fire zones) and to create completely new fire zone consisting of some storage areas with a highfire load density.
4. For the fire safety of structural system: The need to provide the required fire resistance R30 of the structure in the whole plant has been indicated. As the optimal solution it was suggested to separate the office part of the building from warehouses with a high level of fire load density as pretty individual fire zones. In all these areas structural elements (members and systems) should be protected to the level adequate for this type of fire zones.
5. For water supply: It was recommended to provide water to extinguish a fire from the outside by building a fire-fighting water tank.
6. For fire protection equipment: It was recommended to install inside the premises a fire alarm system that will protect the entire facility. Optional connection with the monitoring system will optimize, in case of emergency, the fire department rescue teams travel time.

## 21.7 CONSEQUENCES OF THE CHOSEN SOLUTION ON THE WHOLE LIFECYCLE OF THE BUILDING

Due to technical provisions the fire-fighting equipment should be maintained in accordance with the manufacturer's instructions but not less frequently than once a year. Both the fire alarm system and fixed fire-fighting systems require regular controls exercised during specified periods of time and involving inspections and some specific activities weekly, monthly or every year. Water fire-fighting reservoir and related equipment are also subject to maintenance (every year) but also require to continuously monitor their technical condition. This generates additional costs that appear periodically.

## 21.8 CONCLUSIONS

The presented case demonstrates that during the whole investment process, there may occur some real problems when trying to obtain an administrative decision authorizing the use of the building, despite the seemingly simple and uncomplicated function of the building. Worrying is the fact that, despite the thoughtful and tight system of control and supervision at both the design and implementation phases, there are still problems with the successful completion of the investment which reconciles directly in the interest of the investor.

The authors of this study want to pay a particular attention to the trap that creates a building design based on the formal requirements conditioned by the fire-load density parameter.

The present case shows clearly, that the average fire-load density especially in fire zones and/or premises of large areas can lead to significant underestimations of building fire safety and result in incorrect assessment of the construction located in areas of high fire-load density levels.This problem is not generally discussed in the legal regulations.

There is also no need to prove that the fire load, which, paradoxically, is the greatest in areas with a high probability of fire determines the possibility of structure to survive in fire conditions.

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# 22 FIRE DESIGN OF A NEW FACTORY BUILDING IN ATHENS

Summary

The paper concerns the fire design of the new industrial building in Athens. It is a new, one storey steel building with a total area of about 50.000 m$^2$. The steel structure house the factory plant, customer services and warehouse. The building has been designed against fire, according to the current national regulations for the fire-protection of structures. All the active and passive protection measures are considered and the required fire-resistance of structural members is obtained through fire-protection materials.

In the present paper an alternative approach for the fire design of the structural members is also presented. New cross-sections of the steel members are calculated so that the required fire-resistance is achieved without any protection materials. This alternative fire design is based on the ISO-fire curve and to the regulations of Eurocode 3 – Part 1-2. The two solutions are compared with respect to the total costs.

## 22.1 DESCRIPTION OF THE BUILDING

This case study refers to a new industrial building. The aim of the project is the construction of a multi-area building, combining the production, the logistics centre and the customer service, in order to provide integrated services to the public. The building is going to be placed in Attiki, very near to the city of Athens in Greece. The total area of the one-storey steel structure is 56040.78 m$^2$ while the maximum height is 12 m. It is estimated that the industry will offer workspace for about 300 persons.

The usage of the building is multiple, as it combines an industrial part and an office area. The industrial sector can be divided to three different parts, the inbound, the production and the logistics area. The production zone, of total area equal to 14098.11 m$^2$, accommodates all the appropriate facilities for the manufacturing and the quality control of the products. The inbound zone, of total surface equal to 3482.52 m$^2$, is a part of the overall production area. The inbound building, of total area 10957.35 m$^2$, consists of two different halls that are used for the storage of the products. The customer service building, of total area equal to 3460.40 m$^2$, hosts offices, meeting rooms, storage place, workshop for the electronic devices, kitchen area, restaurants for the employees, sales department and reception hall for the customers. Finally, the Mechanical-Engineering building (M&E building) is a reinforced concrete structure, which houses all the mechanical equipment. It is the area the water tanks are installed. Fig. 22.1 illustrates the plan view of the industry.

The complex of the buildings has three access points from public roads. Two of them are dedicated to staff and visitors while the third one serves the heavy good traffic and it can be used by the fire-brigade vehicles.

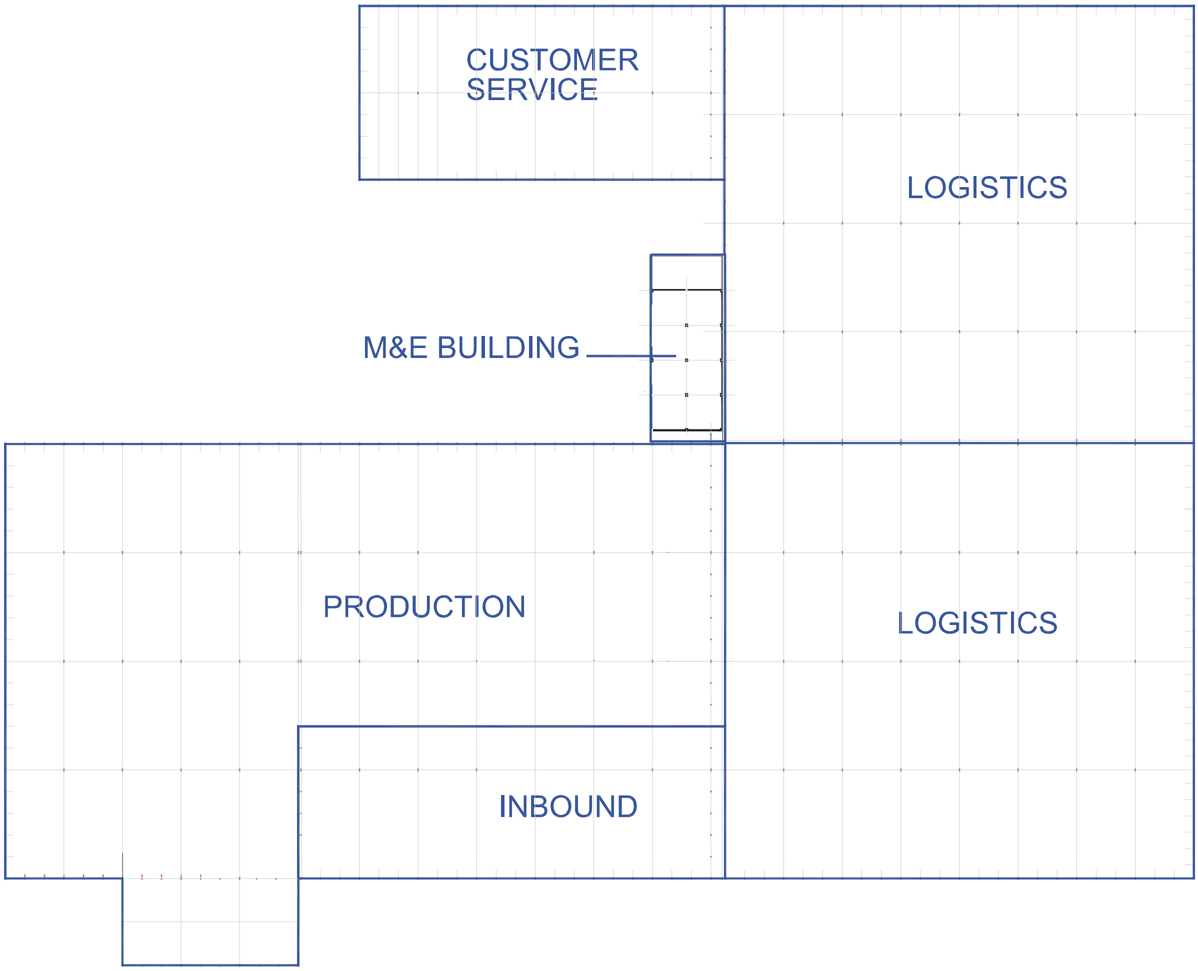


Fig. 22.1 Plan view of the building

### 22.2 FIRE PROTECTION REQUIREMENTS ACCORDING TO THE NATIONAL REGULATIONS

According to the national guidelines, the fire-protection design of the industrial building is based on the P.D. 71 “Regulations for the fire-protection of buildings” (FEK 32, issue A/17.2.1988) and specifically on article 11 which is referred to industry and storage buildings. The regulations specify that, the industrial and storage buildings should be classified according to the risk of the occurrence of the fire event or according to the fire load density. Specifically three categories are defined that are called Z1, Z2 and Z3. The previous are corresponding to small, medium and high risk buildings respectively. The fire-protection measures that are taken into account are strongly dependent on the category of the building. In this case study, the production building is coded as "Construction of home and professional appliances", that is a Small Risk Industry which corresponds to Z1 category. The warehouse building fits to Z2 category due to high density of the fire load. The required fire-resistance time is dependent on the existence or not of sprinkler systems and on the number of storeys. Even though the industry is one-storey ground-level building, it is examined as multi-store due to the existence of mezzanines in the Production and the Warehouse area. Tab. 22.1 presents the required fire-resistance for the buildings.

Tab. 22.1 Required fire-resistance time

| Building | Classification | Required Fire-resistance |
|---|---|---|
| Production | Z1 | 30 mins |
| Inbound | Z1 | 60 mins |
| Logistics | Z2 | 90 mins |
| Customer service | Z1 | 60 mins |

Another important issue that is considered during the fire design of the building is the size of the fire compartments. Specifically, limitations should be taken into account considering the maximum area and the volume of the fire compartment as well as the length of the real and the direct escape roots. In this case study, difficulties are arising on the application of the previous limitations, due to the large size of the building and the specialities of its usage.

According to the regulations, the maximum area of the fire compartment is 5000 $m^2$ considering the case of the production area which is characterised as industry (category Z1). This area can be enlarged by the factor of 2.5, if the appropriate sprinkler system is going to be used. Also the permitted area can be scaled by the factor of 1.5, if the approach of the fire-fighting vehicles is assured by an access road on the perimeter of the building. Taking into account the increasing coefficients the maximum area of the fire compartment is calculated equal to 18750 $m^2$. If the building is considered as one-storey structure, which is the most convenient scenario, the maximum permitted volume of the fire compartment is equal to 28000$m^3$. According to the aforementioned, the height of the building should be 1.49 m, which is extremely paradox. Taking into account the expected height of the building (h=11m), the maximum allowed area of the fire compartment is calculated equal to 2545 $m^2$. This means that the production area, should be divided into 6, at least, different fire compartments, but this is not feasible for the function of the industry.
Moreover, following the corresponding guidelines for the warehouse, the maximum permitted area of the fire compartment is 15000 $m^2$ while the maximum permitted volume is defined equal to 15000 $m^3$. Taking into account the geometric characteristics of the building, the warehouse area should be divided into 16, at least, different fire compartments or the maximum height of the building should be 1.00 m. None of the previous is reasonable, taking into account the proper function of the logistics centre. Considering the

customer service and the inbound area, the dimensions of the fire compartments are mainly defined from the limitations of the escape roots. Fig. 22.2 presents the arrangement of fire compartments according to the national regulations for the fire design of the industry.

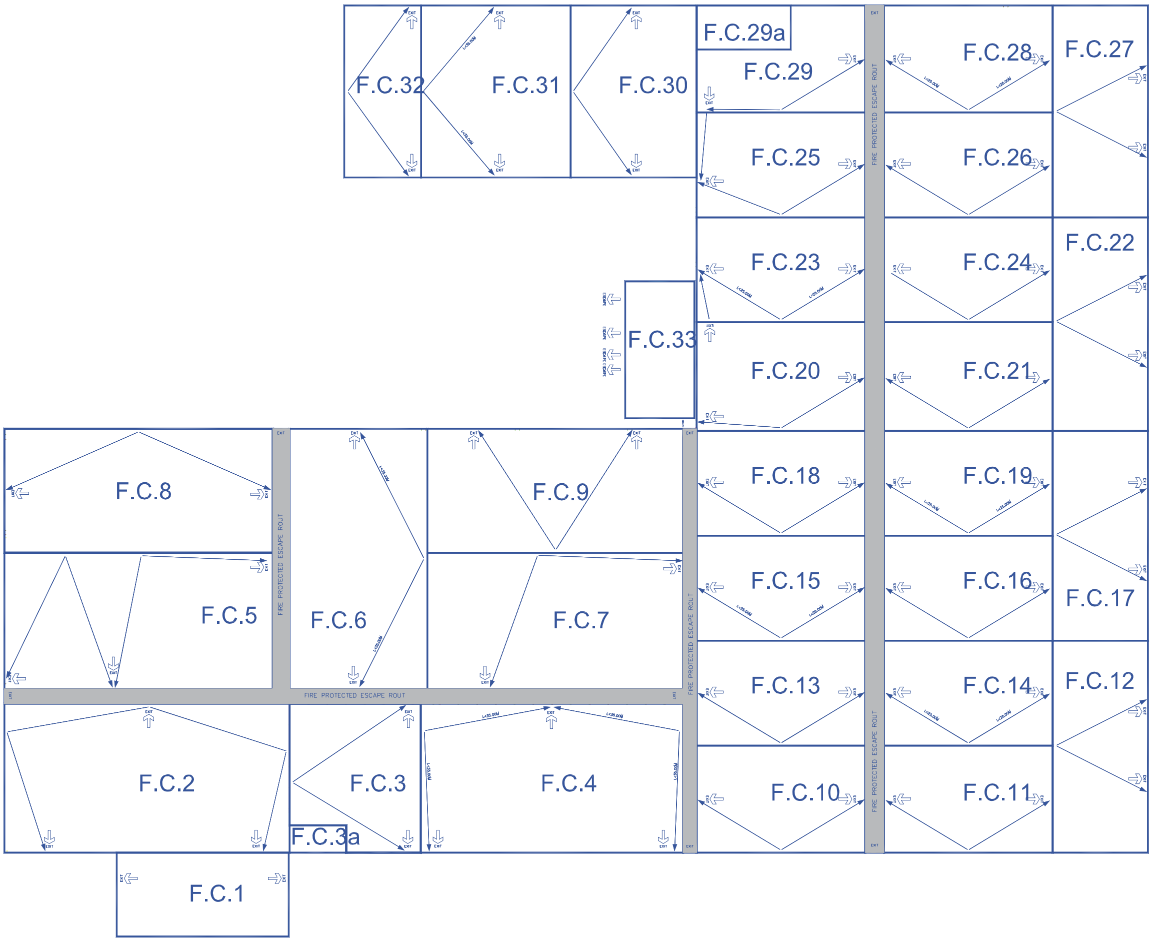


Fig. 22.2 Fire-compartments according to the national regulations for the fire-protection

Concluding, in this case study, the application of the national guidelines regarding the F.C. is impossible. Therefore deviation from the existing rules was asked from the authorities. Tab. 22.2 presents the characteristics of the final fire compartments while Fig.22.3 illustrates the new arrangement according to the deviation.

Tab. 22.2 Characteristics of the final fire compartments

| Fire-Compartment | Use | Area ($m^2$) | Height (m) | Volume ($m^3$) |
|---|---|---|---|---|
| FC 1 | Production | 15133.6 | 11 | 155.079 |
| FC 2 | Inbound | 3706.52 | 11 | 380307 |
| FC 3 | Logistics1 | 10957.35 | 11 | 120530 |
| FC 4 | Logistics2 | 10973.65 | 11 | 120710 |
| FC 5 | Customer service | 4300 | 11 | 38064 |

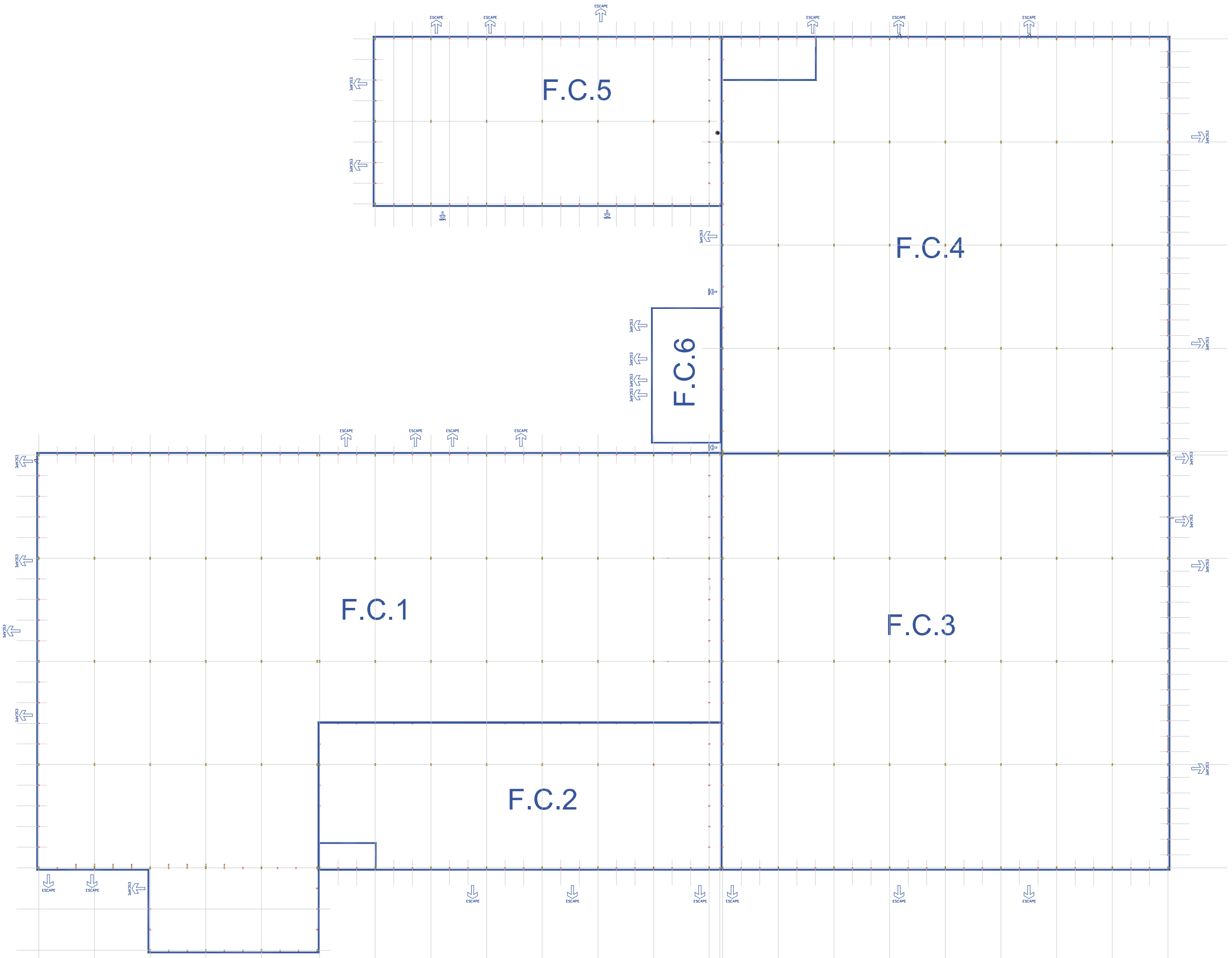


Fig. 22.3 Fire-compartments according to the deviation from the existing rules

## 22.3 STRUCTURAL FIRE DESIGN OF BUILDING

The required fire-resistance of the structure can be fulfilled either through fire-protection materials (fire-proof paintings, fire-resistant boards etc,) either by using the appropriate cross-sections of the members. Here, the study will be focused on the structural fire design of the Customer Service building. Specifically two different approaches are examined. The first approach is based on the use of the fire-protection materials in order to achieve the required fire-resistance. The second approach proposes alternative cross-sections and the fire-resistance is achieved without fire-protection materials. Finally, the effectiveness of the two different approaches is compared, in terms of financial cost.

Fig. 22.4a presents the plan view of the building and a typical frame of the steel structure is illustrated in Fig. 22.4b.The typical frame is repeated every 13.5 m.

It should be noted that the steel structure is primary designed against earthquake events and the fire design is the second step. The seismic design is performed according the national regulations [EAK 2000]. The cross-sections that are calculated during this stage are summarized in Table 3.

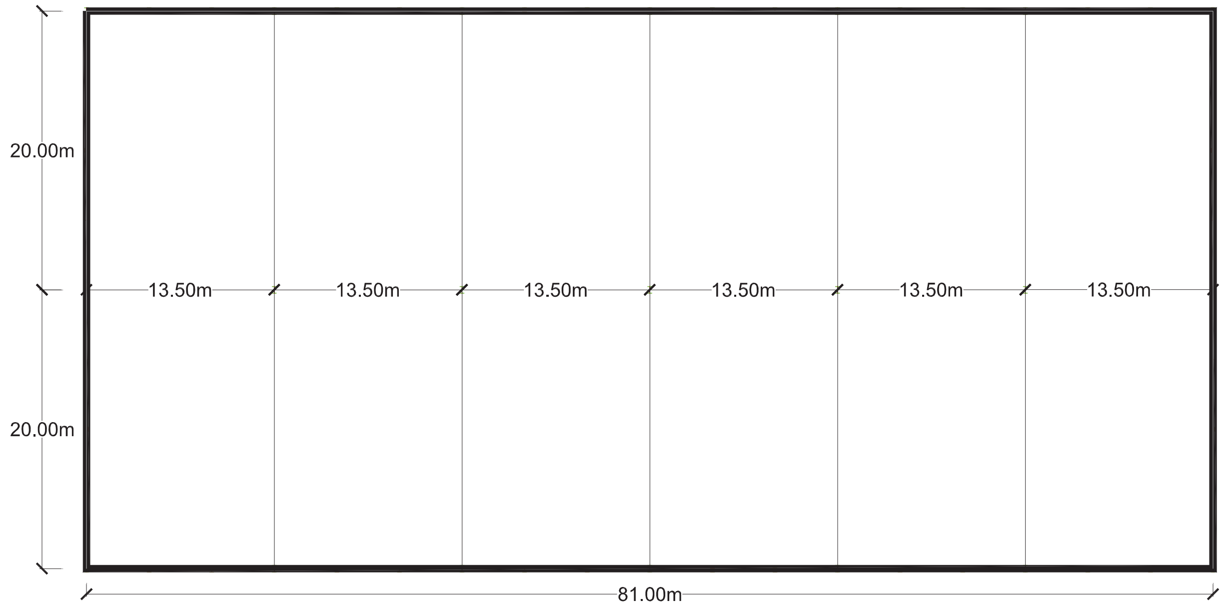


Fig. 22.4a Plan-view of the Customer Service building

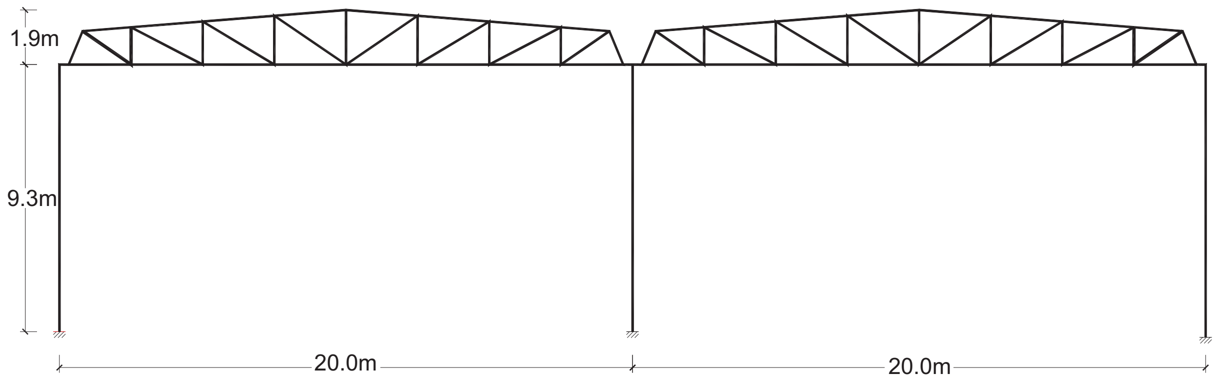


Fig. 22.4b The typical frame and the cross-sections coming from the seismic design

**22.3.1 Structural fire design of building -First approach**

According to the first approach, the required fire-resistance time, that is indicated equal to 60 minutes, is achieved through fire-proof painting. Specifically the seismic design is the first step and the second step is the estimation of the characteristics of the required fire-proof painting. The thickness of the painting and consequently the total financial cost is dependent on the fire-resistance time and on the dimensions of the cross-sections. Tab. 22.3 presents the required thickness of the painting for the structural members including the typical frame and the purlins. The calculations are conducted for fire-resistance time equal to 30 minutes and 60 minutes. It is noted that the total financial cost that is calculated includes the cost of the painting and the working cost for the application of the painting taking into account the Greek commercial market.

Tab. 22.3 Results of the first approach

| Seismic design | | | Fire-design | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | R30 | | | R60 | | |
| Section | Length (m) | Self-weight (kN) | Thickness of the painting (μm) | Weight of the painting (kg) | Cost (€) | Thickness of the painting (μm) | Weight of the painting (kg) | Cost (€) |
| HEB500 | 27.9 | 52.3 | 265 | 36.2 | 333 | 504 | 68.8 | 451 |
| HEA220 | 81.9 | 41.3 | 330 | 78.2 | 634 | 1026 | 243.0 | 1227 |
| HEA120 | 243.0 | 48.3 | 368 | 139.5 | 1951 | 1933 | 737.7 | 4086 |
| RHS120/6.3 | 8.2 | 27.4 | 860 | 140.4 | 748 | 2000 | 326.5 | 79 |
| RHS100/5 | 24.7 | 29.8 | 1135 | 157.6 | 833 | 3560 | 380.5 | 296 |
| QHS80/3.6 | 315.2 | 27.2 | 1650 | 383.4 | 2269 | 5720 | 1329.2 | 5673 |
| RHS80/5 | 6 | 0.7 | 1135 | 5.0 | 25 | 3560 | 15.7 | 63 |
| RHS70/3 | 16.6 | 1.4 | 1150 | 15.6 | 73 | 5090 | 50.6 | 199 |
| RHS60/4 | 23.8 | 1.7 | 1150 | 15.1 | 74 | 5090 | 66.9 | 260 |
| QHS50/4 | 142.2 | 8.2 | 1150 | 75.4 | 522 | 5220 | 342.0 | 1482 |
| QHS40/2.9 | 124.6 | 4.2 | 1600 | 73.5 | 440 | 2290 | 105.2 | 554 |
| TOTAL | | 214.7 | | 14006.3 | 6421 | | 46033.3 | 14223 |

### 22.3.2 Structural fire design of building -Second approach

In this approach the required fire-resistance time is achieved without fire-protection materials and the calculation of the alternative cross-sections of the structural members is based on Eurocode 3 Part 1-2 using the ISO fire curve. It is indicated that no thermal analysis takes place and that the temperature is assumed to be uniform in the cross-sections and along the members. Specifically, the temperature of the structural members is calculated according to section 4.2.5 of EN 1993-1-2 and depends on the geometric characteristics of the cross-sections taking into account the corrections about the shadow effects. The target is to estimate the appropriate cross-sections of the structural members in order to resist in fire for 30 and 60 minutes respectively. The starting point of the calculation deals with the cross-sections that are coming from the seismic design. This calculation is actually an iterative procedure and the subsequent steps are the following:

**Step 1**: Calculation of the temperature of structural members of the typical sub-frame and the purlins, at the desired time *t*

**Step 2**: Static analysis for the fire combination $G+\psi_2 Q$

**Step 3:** Checking if the cross-sections are adequate for the fire combination at the desired time *t*. If they are adequate the calculation is finished. Otherwise step 4 follows.

**Step 4:** Calculation of the new cross-sections

**Step 5:** Repeat Step 2, 3 and 4 for the new cross-sections

The results of the iterative procedure are presented in Tab. 22.4, indicating the increase of the self-weight of the steel structure in relevance with the fire-resistance time. It is observed that in the case of 30 minutes required fire-resistance the self-weight of the structure is increased 34.75%, while the corresponding value for the R90 requirement is 76.33%.

Tab. 22.4 Variation of the self-weight of the structure according to the design requirements

| | Seismic design | R30 | R60 |
|---|---|---|---|
| Self-weight of the structure | 214.72 kN | 289.33 kN | 378.68 kN |

## 22.4 COMPARISON OF THE ALTERNATIVE APPORCHES

The alternative approaches are compared in terms of financial cost. The comparison specifies that the most economical approach seems to be the first one which uses the fire-proof paintings (Tab. 22.5). The results of the second approach can be reconsidered if global elastic-plastic analysis of the steel structure is taken into account. It is expected that in this case the outcomes of the second approach would result to reduced financial cost, comparing with the results of the same approach in this study.

Another issue that should be noted is that the second approach is based on the ISO fire curve as it is indicated in the guidelines. Alternatively, this approach can be based on parametric fire curves, zone models or CFD temperature results. This is a critical parameter for the variation of the results of this approach.

Tab. 22.5 Comparison of the alternative approaches

| | | Self – weight | Financial cost | Fire-proof painting | Financial cost | Total financial cost |
|---|---|---|---|---|---|---|
| **R30** | Approach 1 | 21472 kg | 34355 € | 842 kg | 6421 € | **40776 €** |
| | Approach 2 | 28933 kg | 46293 € | - | - | **46293 €** |
| **R60** | Approach 1 | 21472 kg | 34355 € | 3011 kg | 14229 € | **48584 €** |
| | Approach 2 | 37862 kg | 60580 € | - | - | **60580 €** |

## 22.5 SUMMARY

This paper presents the fire design of an industrial building that is going to be placed in Athens. Primary, the fire design requirements, according to the national regulations, are presented. It is concluded that in this case study, the guidelines concerning the fire compartment limitations are not applicable. Therefore, the final arrangement of the fire-compartments, according to the deviation from the existing rules, is presented. In the second part of the paper, two alternative approaches for the structural fire-design of the Customer service building are studied. The results indicate that the most economical approach is the one that is based on the use of fire-proof materials.

Acknowledgement

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# 23 APPLICATION OF STRUCTURAL FIRE ENGINEERING TO OPEN AND CLOSED CAR PARKS OF C.A.S.E. PROJECT FOR L'AQUILA

Summary

The Fire Safety Engineering (FSE) is a multi-discipline aimed to define the fire safety strategy for buildings under fire conditions, in which structural stability and control of fire spread are achieved by providing active and/or passive fire protection. In the following the aspects of FSE for the structural safety checks in case of fire (Structural Fire Engineering) are shown with reference to Italian and European standards.

FSE requires the choice of a performance level, the definition of design fire scenarios, the choice of fire models and several numerical thermo-mechanical analyses. The information provided by a significant research, performed in Europe for open and closed car parks, are used to apply the FSE to the car parks of the new buildings of the C.A.S.E. Project for L'Aquila, characterized by steel columns supporting the seismically isolated superstructure. The results of the application of the FSE approach will be reported and discussed.

## 23.1 INTRODUCTION

According to ISO/TR 13387-1, the "Fire Safety Engineering" (FSE) is the application of engineering principles, rules and expert judgement based on a scientific assessment of the fire phenomena, the effects of fire and both the reaction and behaviour of peoples, in order to:

- save life, protect property and preserve the environment and heritage;
- quantify the hazards and risks of fire and its effects;
- evaluate analytically the optimum protective and prevention measures necessary to limit, within prescribed levels, the consequences of fire.

Current Italian and European codes allow the use of a performance approach through the concept of Fire Safety Engineering. The temperature distribution within the elements and the mechanical and geometric nonlinear structural response are taken into account in the fire performance approach.

The Directive 89/106/CEE on Construction Products of the European Community introduced the definition of the requirement of “safety in case of fire” in Europe, which is the base for the application of the Fire Safety Engineering. This requirement, implemented in the National Codes of European member countries, is explained by achieving the following five objectives:

- the load-bearing capacity of the construction can be assumed for a specific period of time;
- the generation and spread of both fire and smoke within the works is limited;
- the spread of fire to neighbouring construction works must be limited;
- occupants have to be able to leave the works or be rescued by other means;
- the safety of rescue teams must be taken into consideration.

The results of each application of the performance approach to the fire safety should be evaluated through the analysis of the achievement of these objectives.

The Fire Safety Engineering allows a more accurate adjustment of the safety measures at specific risk of the building through qualitative and quantitative criteria (namely acceptance criteria) which have been agreed with the building approval authority and hence form an acceptable basis for assessing the safety of a building design.

The European codes for structural fire safety is represented by the “Fire Parts” of Structural Eurocodes.

In Italy, the new Technical Code for Constructions has been published in 2008. For the first time in Italy, the fire action is introduced within the definition of the actions on constructions, as an “exceptional load”. The document defines the performance safety levels of buildings according to the safety objectives required by the Directive 89/106/CEE. The Italian Technical Code for Constructions defines five safety performance levels depending on the importance of the building, which establish the damage level that can be accepted. These rules define the fire structural performance requirements and refer to specific technical codes issued by the Italian Ministry of Interior for all activities under the control of the National Fire Brigades, see (Ministry of Interior 2007a and 2007b). The regulations are basically prescriptive and concern several types of building use. However, the performance based fire design and advanced calculation models may be applied either in the lack of prescriptive rules or in the case of “derogation” with respect to prescriptive rules. The performance based design has to developed according to Decree of the Ministry of the Interior of 09/05/2007, see (Ministry of Interior 2007b), titled “Direttive per l’attuazione all’approccio ingegneristico alla sicurezza antincendio”. The fire design, according to D.M. 09/05/2007, summarized in Fig. 23.1, is divided in two stages: the first is preliminary analysis, i.e. qualitative analysis, while the second is quantitative analysis. Between the first and second stage, the approval of design fire scenarios by Italian Fire Brigade (Vigili del Fuoco) is needed. Finally, it is important to note that in the current Italian code the performance-based approach does not replace the prescriptive one, but both the approaches coexist. The technical solutions imposed by the prescriptive approach remain one of the possible ways that the designer may choose for the structural fire design.

The following describes the application of FSE (namely the structural behaviour in fire situation) to the car parks in the new buildings of the "C.A.S.E. Project for L'Aquila". This Project was developed in L'Aquila (province of Abruzzo, Italy), after the seismic event of 06/04/2009, in response to the housing emergency. The car parks, placed at the ground floor of the buildings, are mainly built with steel columns that support the seismically isolated superstructure. The Italian prescriptive code provides, for car parks, a fire resistance class for the load-bearing criterion of 90 minutes in standard fire exposure (R90). However, for obtaining the fire resistance class R90 the adoption of protective coatings of steel columns is needed, for which a continuous and accurate maintenance is required: in fact, there is a high possibility of accidental damage of the protective coatings in case of impact with the cars. Moreover, the possibility of damage becomes elevated when a series of acts of vandalism takes place, for example if the car parks are easily accessible and not controlled. Because of the uncertainties on the effectiveness of coatings maintenance, in such cases, their use is not recommended.

Therefore, the lack of protective coatings on steel columns and the structural safety during the fire exposure can be evaluated through the application of performance-based approach, which allows to assess, in a more complete and reliable manner, the structural response with reference to the fire scenarios that can realistically occur.

## 23.2 CASE STUDY: CAR PARKS

### 232.1 Building description: analysis of the structural characteristics

Each residential building is built on a seismically isolated plate, with dimensions equal to $21 \times 57$ m$^2$ about, capable of supporting a three-storey building with dimensions in plant equal to $12 \times 48$ m$^2$ about, in addition to the stairs. The buildings (superstructures) are different for architectural and constructive elements; the structures are built with wood materials, reinforced concrete or steel. Each isolated plate (with height of 50 cm) is sustained by steel columns (with height of 260 cm) by the isolation system. In this area, below each seismically isolated plate, the parking (Fig. 23.2) for about 34 cars are contained. In order to distribute the actions on the reinforced concrete foundation plate the columns are allocated on a 6 m × 6 m grid. The dimensions in plant of the compartment are equal to $22 \times 58$ m$^2$; in fact the outside walls, when present, are mismatched 50cm with respect to the vertical projection of the edge of the seismically isolated plate.

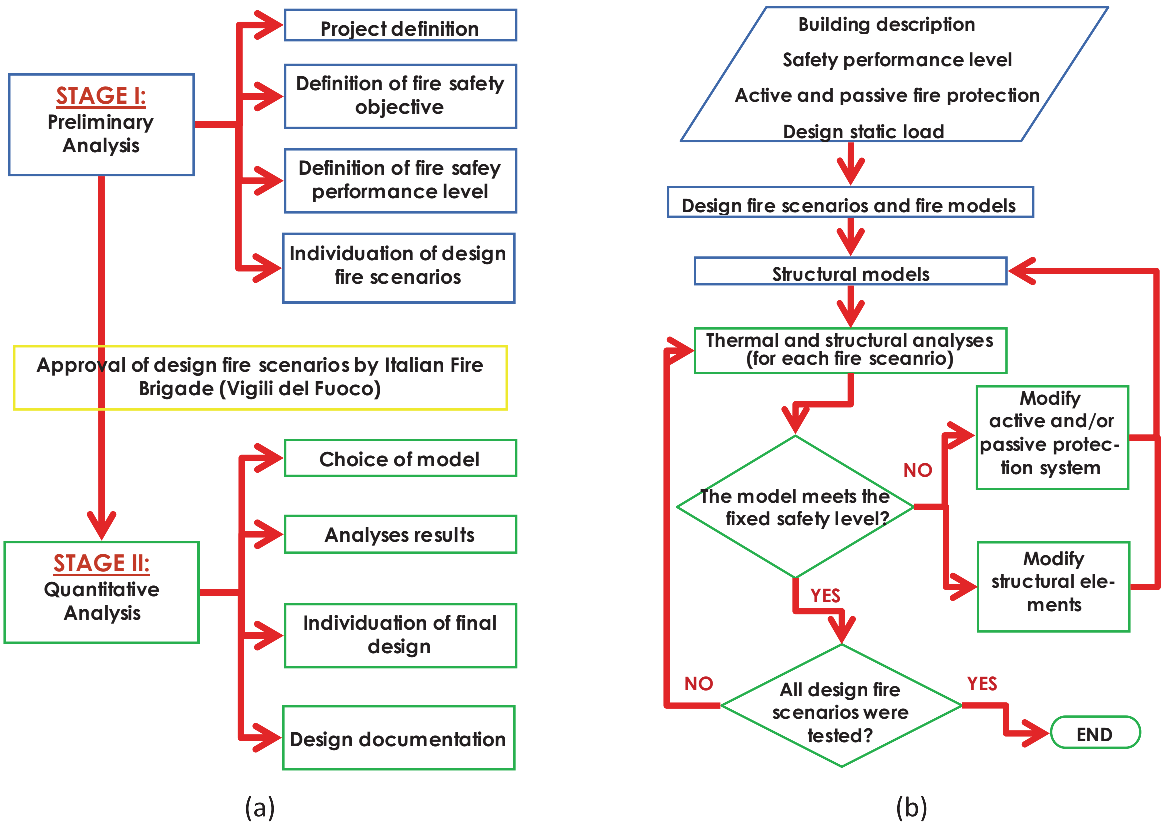



Fig. 23.1 Fire Safety Engineering: Italian code process according to Decree of the Ministry of the Interior of 09/05/2007, see (Ministry of Interior 2007b)

The steel columns are a circular hollow steel section with a capital at the top; this latter is useful a) for transferring, through the isolator unit, the load between the column and the seismically isolated plate and b) as a structure of contrast to the operations of substitution of the isolator unit. The parking area can be fully open on the four sides or partially closed on one or more sides. Therefore, among the various examined cases are present both open car parks and almost completely closed, as well as several intermediate cases.

### 23.2.2 Choice of safety performance level

In this case study, the objective of fire safety design concerns the mechanical resistance and stability, in fire situation, of the primary structural elements in the zone below the seismically isolated plate. In order to attain this objective, based on the superstructure use (residential buildings), it is sufficient to guarantee that the structures fire resistance requirements for a period consistent with the emergency management are respected (according to performance level III of the Italian Code, see (Ministry of Infrastructure and Transport, 2008)). Nevertheless, in this case a limited damage after the fire exposure has been also required. The damage is quantified in terms of relative vertical displacements between the top of two adjacent columns: in order to limit the finishing damage in the superstructure, the relative vertical displacement must not exceed the limit value, chosen cautiously equals to L/200 (5.0 ‰), where L is the distance between two adjacent columns (L= 6000 mm).

Fig. 23.2 Parking zone

### 23.2.3 Choice of the active and passive fire protection systems

No specific protection systems (active and/or passive) are provided.

### 23.2.4 Static and fire design load calculation

The Italian and European codes (Ministry of Infrastructure and Transport, 2008 and EN 1991-1-2) classify the fire as an exceptional load, so the fire design load combination is defined by:

$$F_d = A_d + G_{k1} + G_{k2} + \sum_{i=1}^{n} \psi_{2i} \cdot Q_{ki} \qquad (1)$$

where $G_{k1}$ is the characteristic value of permanent structural load; $G_{k2}$ is the characteristic value of permanent non structural load; $\psi_{2i} \cdot Q_{ki}$ is the quasi-permanent value of a variable action *i*; $A_d$ is the design value of an exceptional action.

Because of the great variability of the superstructure structural type, the fire structural analyses have been carried out, for simplicity and for the benefit of safety, with reference to the maximum combination of exceptional load (maximum axial load on each column equal to 1800 kN).

The design fire load density is closely linked to the type of cars which may be found in the car parks. The cars can be classified according to the thermal energy that can release during the fire. In it is reported the classification of cars (circulating in the period 1995-1998) based on the calorific potential of cars. This classification can be found the final report of CEC agreement 7215-PP/025, concerning a research activity conducted by CTICM (France), Profil-Arbed Recherches (Luxembourg) and TNO (Netherlands) and concluded in 2001. The cars were classified in five categories according to their calorific potential value. In relation to currently circulating cars, it is possible to classify how cars belonging to an inferior or equal category to that of "category 3" (the one having a cylinder capacity not exceeding 2000cc), while those with cylinder capacity upper than 2000cc belong to the "categories 4 and 5" (Tab. 23.1).

The percentage of vehicles, circulating in Abruzzo at the date of 31/12/2008, of cylinder capacity exceeding 2000cc is equal to 6.6% of the total vehicles (from statistics of A.C.I. - Italian Automobile Club). Therefore, because each car park has a maximum capacity of 34 vehicles, the percentage of vehicles with cylinder capacity exceeding 2000cc corresponds to 2 vehicles on 34. Moreover, assuming "category 3" as the category representative of the circulating cars, it is possible to assume the contemporary presence of 32 vehicles of category 3 (calorific value equals to 9500 MJ), and 2 cars of superior category or 2 commercial vehicles; for the scope of the analyses, it refers to commercial vehicles (VAN) with calorific value of 9500 MJ containing 250 kg of highly inflammable material (calorific value of 40 MJ/kg), for a total of 19500 MJ.

Tab. 23.1 Classification of cars, see (CEC Agreement, 2001)

| Type | Category 1 | Category 2 | Category 3 | Category 4 | Category 5 |
|---|---|---|---|---|---|
| Peugeot | 106 | 306 | 406 | 605 | 806 |
| Renault | Twingo-Clio | Mégane | Laguna | Safrane | Espace |
| Citroën | Saxo | ZX | Xantia | XM | Evasion |
| Ford | Fiesta | Escort | Mondeo | Scorpio | Galaxy |
| Opel | Corsa | Astra | Vectra | Omega | Frontera |
| Fiat | Punto | Bravo | Tempra | Croma | Ulysse |
| Wolkswagen | Polo | Golf | Passat | // | Sharan |
| Theoretical energy | 6000 MJ | 7500 MJ | 9500 MJ | 12000 MJ | |

Therefore, once defined the distribution of cars, it is possible to determine the design fire load density. This latter can be evaluated from the characteristic fire load density, defined as sum of thermal energies, which are released by combustion of all combustible materials in a space, per unit area related to the floor area. In this case the specific fire load density is:

$$q_f = \frac{H_{tot}}{A_{tot}} = \frac{32 \cdot 9500\,\mathrm{MJ} + 2 \cdot 19500\,\mathrm{MJ}}{1276\,\mathrm{m}^2} = 268.08\ \mathrm{MJ/m^2} \tag{2}$$

Finally, according to EN1991-1-2, the design fire load density can be evaluated as:

$$q_{f,d} = \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \cdot q_f = 1.4 \cdot 1.0 \cdot 0.9 \cdot 268.08\ \mathrm{MJ/m^2} \cong 340\ \mathrm{MJ/m^2} \tag{3}$$

where $\delta_{q1}$=1.4 (factor taking into account the fire activation risk due to the size of the compartment), $\delta_{q2}$=1.0 (factor taking into account the fire activation risk due to the type of occupancy) and $\delta_n$=0.9 (factor taking into account the different active fire fighting measures i) are defined according to Italian code (Ministry of Interior, 2007a).

**23.2.5 Fire design scenarios and Fire model**

The fire scenario is significantly affected, among other things, by the geometry and ventilation conditions of the compartment. As regards the evaluation of number of vehicles involved in the fire and the timing of fire initiation by a car to adjacent one, reference is made to the informations from the final report of CEC agreement 7215-PP/025, where are reported the results of real fires in car parks and full scale tests conducted in Vernon (France), both in the presence of free ventilation and with limited ventilation. These results have allowed the drafting of guidelines INERIS currently used in France for the definition of fire scenarios in car parks according to Decree of French Ministry of Interior of 9 may 2006, see (Arrête, 2006).

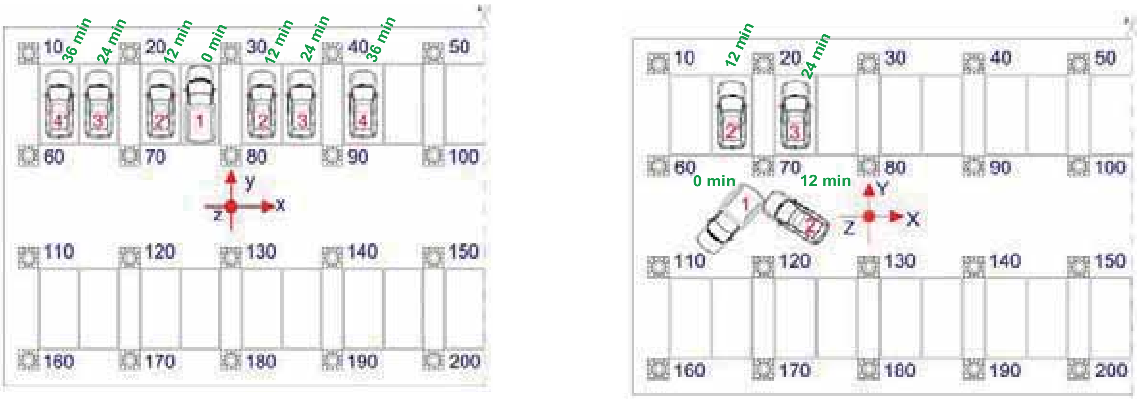
It is necessary to distinguish the car parks open on all their sides by those partially open (openings limited or absent on one or more sides). The presence of natural ventilation in open car parks does not allow the achievement of the flashover conditions: for this reason the phenomenon remains for the entire fire duration of "pre-flashover" type. In these conditions a limited number of vehicles, near the ignition source, burn. In partially open car parks, instead, it is possible that the fire involved all of the cars.

Therefore, the identification of the more dangerous fire scenarios for the structural stability is to define the position and the number of cars that may be involved in the fire and cause the more dangerous thermal action, between those realistically conceivable, for the supporting structure building.

By applying the criteria proposed in the aforementioned guidelines to *car parks open on all sides* the types of distribution of the cars described in Tab. 23.2 are chosen, with a fire propagation time from car to adjacent one equals to 12 min. Thanks to the symmetry of car parks' structures, in order to maximize the fire effects, the vehicles are located according to the Fig. 23.3.

Tab. 23.2 Cars distribution for localised fire scenarios (pre-flashover)

| | |
|---|---|
| Type 1 (L1) | 7 vehicles, of which 1 central VAN and 6 cars, that burn with a fire propagation time from car to adjacent one equals to 12 min from the VAN. |
| Type 2 (L2) | 4 vehicles, of which 1 central VAN and 3 cars surrounding a column, that burn with a fire propagation time from car to adjacent one equals to 12 min from the VAN. |



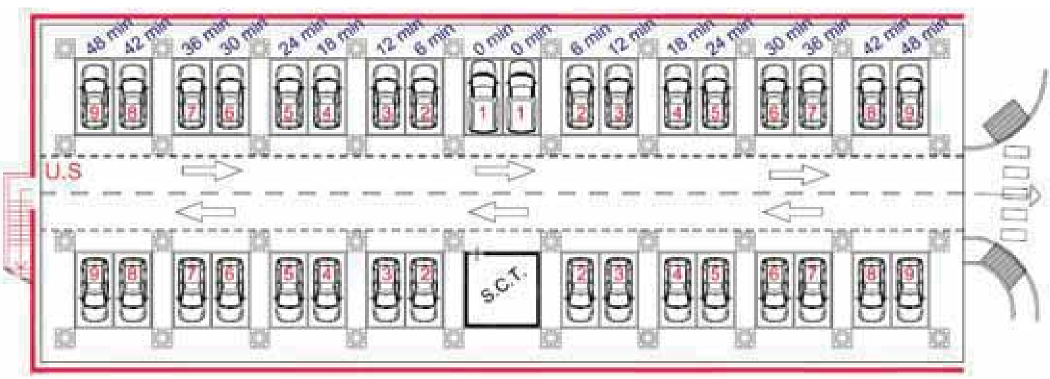

Scenario L1 Scenario L2

Fig. 23.3 Pre-flashover fire scenarios

Instead, with regard to the *partially open car parks* (openings limited or absent on one or more sides), in addition to considering the localised fire scenarios (pre-flashover), there must be considered generalized fire scenarios as well (post-flashover), which involve, in the extreme event that the whole of car space available is occupied, all present vehicles. The time of the spread chosen for this case from a car to adjacent one is 6 min, in agreement with the results of the above experimental full-scale tests with limited ventilation. Therefore, the types of distribution of the cars (with 6% of VAN) described in Tab. 23.3 are chosen, with a fire propagation time from a car to adjacent one equals to 6 min. The vehicles location and the spread time are reported in the Fig. 23.4.

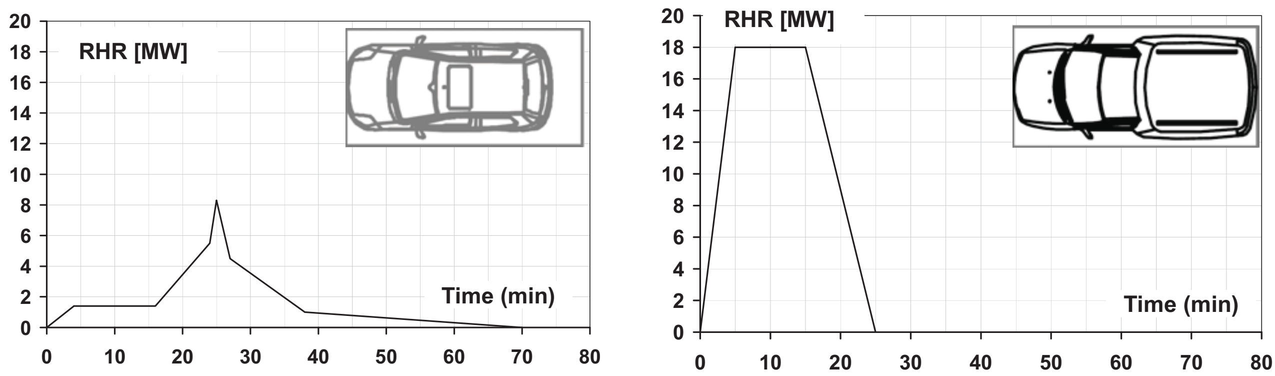
Tab. 23.3 Cars distribution for generalized fire scenarios (post-flashover)

| | |
|---|---|
| Scenario D1 (6% VAN) | 34 vehicles, of which 2 central VAN and 32 cars, that burn with a fire propagation time from car to adjacent one equals to 6 min from the VAN |



Scenario D1 (6% VAN)
Fig. 23.4 Post-flashover fire scenario

For localised fire (pre-flashover) the temperature in the fire flame and plume and the surrounding gas are not uniform, and need to be determined separately. Instead in a post-flashover fire the temperature is assumed to be uniform within the fire compartment. Eurocode 1 Part 1-2 (Annex C, EN1991-1-2) provides a simple calculation method (Hasemi's Method) for determining the thermal action of localised fires of compartments in which the input data is the heat released by combustible products (in this case the single car) as a function of time (namely Rate of Heat Release - RHR). The Rate of Heat Release curves for the single burning car of "category 3" and for a single burning VAN are provided by calorimetric hood tests reported in the final report of the quoted CEC agreement 7215-PP/025. In Fig. 23.5 are reported the RHR curve of a) car of category 3 and b) VAN obtained by fitting the experimental results.




a) Category 3 (car) b) VAN

Fig. 23.5 RHR curves

For the generalised fire scenarios, the temperatures in the compartment are evaluated by the software OZone ver. 2.2 (Cadorin & Franssen, 2003). OZone ver. 2.2 is a zone modelling software (according to the Appendix D of EN 1991-1-2) which the temperature development of the gases within a compartment during the course of a fire allows to evaluate from the input data as: the geometric characteristics of the compartment, the thermal characteristics of the materials of which it consists, the ventilation conditions and the rate of heat release obtained through the overlapping time in the sequence of initiation of the single car RHR curves.

The results of the two fire models (localised fire and generalized fire) is different: in fact, the Hasemi's method gives the heat flux received by the fire exposed surfaces at the level of the ceiling, while the zone model provides the temperature within the compartment.

For the localised scenarios (L1 and L2), in Fig. 23.6 the heat flux received by some significant steel columns are reported.

For the generalized fire scenarios, in Fig. 23.7 are reported the compartment time-temperature curves given from the zone model of the fire scenarios D1. Because of the several possible ventilation conditions for the car parks case study, 7 different ventilations classes (V1,...,V7) were considered. In order to maximize the fire exposure, the structural analyses have been referred only to the ventilation condition V1; in fact, because of the fact that the temperature of the structural elements is mainly dependent on the growth phase and on the maximum fire temperature, the fire with ventilation conditions V1 is the most dangerous one.

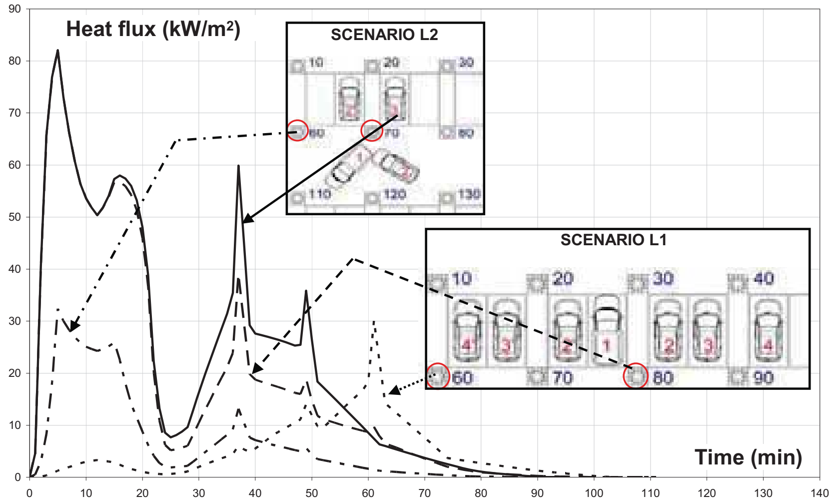



Fig. 23.6 Thermal flux from Hasemi's method

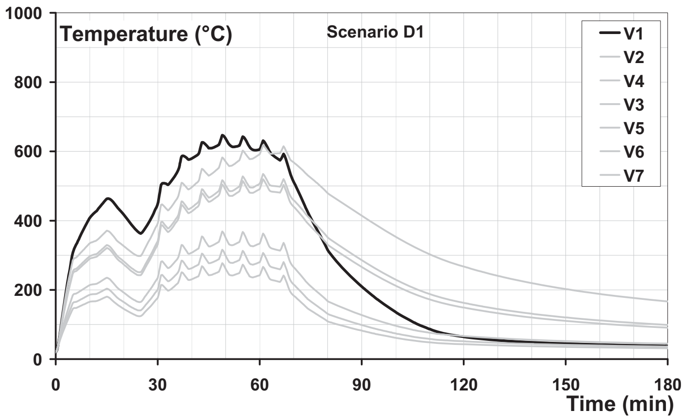
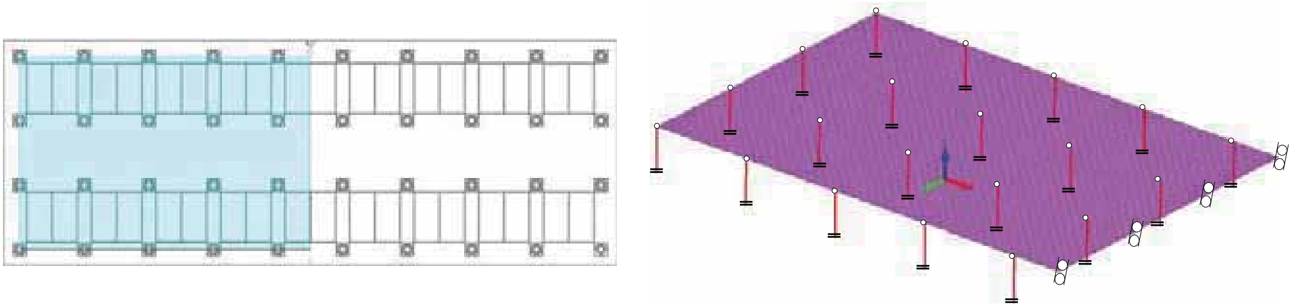



Fig. 23.7 Time-Temperature curves for post-flashover fire

### 23.2.6 Structural model and fire safety assessment

In order to limit the analysis time without compromising the accuracy of the results, the thermo-mechanical analyses, for each fire scenario, have been conducted with the reference to the substructure highlighted in Fig. 23.8 (Nigro, 2010). The substructure extension allows assessing in an appropriate way both the thermal field and the hyperstatic effects induced by different thermal expansions of steel columns and bending of the concrete reinforced slab. Along the edge a constraint is introduced for the horizontal movements in the longitudinal direction and for the rotations around transverse axis. This constraint condition, thanks to the structural symmetry, is fully congruent for the analysis in normal temperature conditions and for the generalised fire scenarios (scenario D1), while it is on the safe side for the other scenarios (localised fire scenarios), maximizing, thanks to the infinite rotational stiffness, the hyperstatic effects induced on the columns by slab thermal curvature. The steel columns are fixed at the base and linked to the superstructure slab with a hinge.



Substructures Static scheme

Fig. 23.8 Thermo-mechanical model of the structure

For each fire scenario, the global thermal-mechanical structural analyses of the substructure in Fig. 23.8 are conducted by using the non linear software SAFIR2007a (Franssen, 2005), developed at the University of Liege (Belgium), which performs the structural analysis under fire conditions. The steel columns are modelled with beam elements with circular cross-section, while the reinforced concrete slab is modelled with shell elements. In addition to the global analysis, for each fire scenario, in order to calculate more accurately the thermal field and stresses distribution in the capitals above the columns and to assess the possible local buckling, a detailed thermo-mechanical analyses has been conducted with reference to the more stressed and heated column. The 3D modelling (Fig. 23.9) have been developed with the finite element software ABAQUS/standard. The thermal exposure conditions were considered according to Fig. 23.6 and Fig. 23.7. The axial load corresponds to the axial load obtained by the global structural analyses.

Fig. 23.9 3D thermo-mechanical model of the column

### 23.2.7 Analyses results

For sake of brevity, the results of structural response in fire situation are reported only with reference to the fire scenario L2, which appears more unfavourable (Fig. 23.10). The maximum temperatures reached in the columns do not exceed 600°C (Fig. 23.10-a). It appears clear that the highest temperatures reached in columns, namely those nearest to the VAN (characterized by a higher calorific potential), reach a maximum temperature of about 580°C in correspondence of a fire exposure time of about 20 minutes. The thermal action produces both in the columns and slabs several thermal expansions. Because of the thermal curvature of the slab the columns axial load increases (Fig. 23.10-b). The axial load is further amplified from the differential thermal elongation (Fig. 23.10-c) of columns, exposed to different thermal conditions, which is constrained from slab shear stiffness.

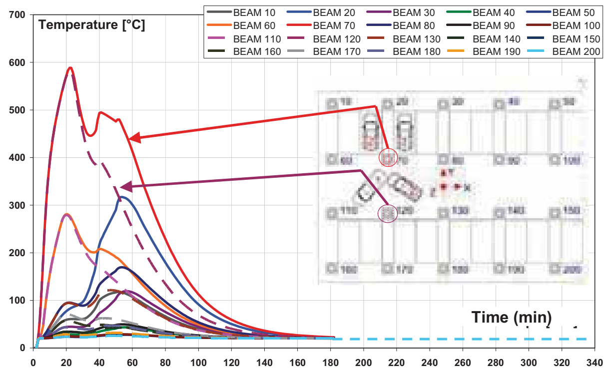
The thermal expansion induced by fire leads to the columns elongation with consequential upwards displacement (Fig. 23.10-c). The displacement reflects, in general, the temperatures trend. However, the reduction of stiffness that the structural elements suffer, if constrained to high temperatures, may lead to a premature reversal development in displacement. In fact, to the column 70 of the scenario L2, the displacements increase until a fire exposure time of about 18 minutes (corresponding to steel column temperature of about 570°C), reversing later on their trends because of the presence of a high axial load and the elastic modulus reduction with the temperature. The shortening is subsequently determined also by the reduction of the temperature which begins after about 20 minutes of fire exposure. By following the almost complete cooling phase of the columns it is possible to notice a small residual deformation; in fact, the final displacement, after the fire exposure, is slightly different from the initial elastic one (namely, before the start of fire).

Moreover, the maximum differential displacement reached between the adjacent columns is very small. In fact, the maximum differential displacement, during the fire exposure, between the column 120 and column 130 is of about 16mm at the time of exposure of about 20 minutes. This value corresponds to 2.6 ‰ (16mm/6000mm), definitely below the limit value of 5.0 ‰ taken as the acceptance criteria.

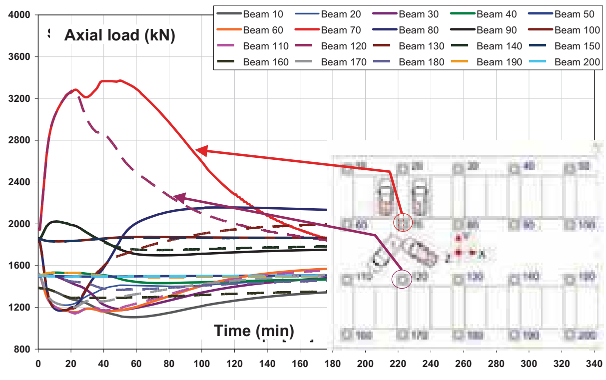
The checks on resistance automatically carried out by the software are also integrated by comparing the axial load during the fire to the axial load resistance of the columns evaluated according to EN1993-1-2. In Fig. 23.10-d is showed that the more loaded column, even when the maximum temperature is reached, still has a significant reserve of resistance.

As regards the detailed analysis of the column, the displacement at the head of column is very similar to those obtained in the global structural analyses (Fig. 23.11). The final displacement is about 5mm in the central area of capital and about 2mm in the tube head: this is due to the plastic strain which has developed in the tube and in the capital (mainly in the zone of load application) during the fire exposure.

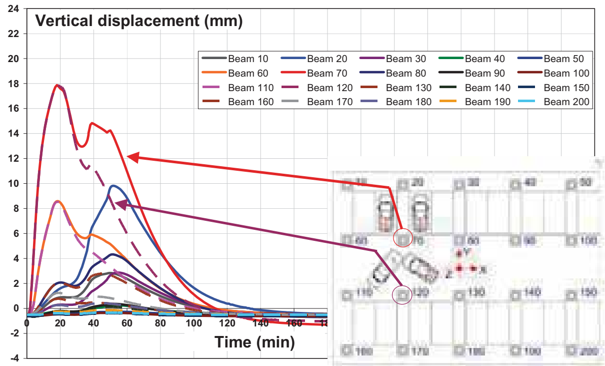
Similar considerations are also valid for the other fire scenarios. Therefore it can be concluded that the structure, and in particular the columns in the absence of any protection system against fire, during the design fire exposure perform an adequate load-bearing capacity, including the cooling phase.



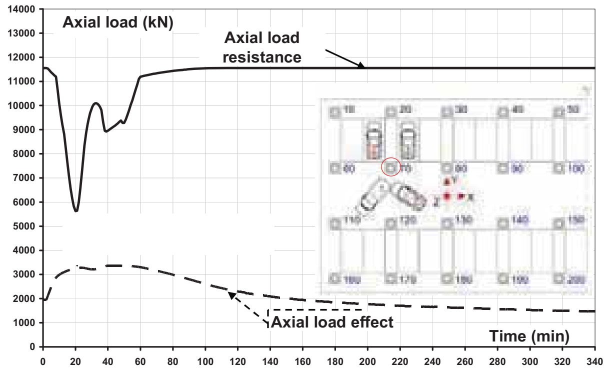

a) Time-temperature curves for the columns




b) Time-axial load curves for the columns

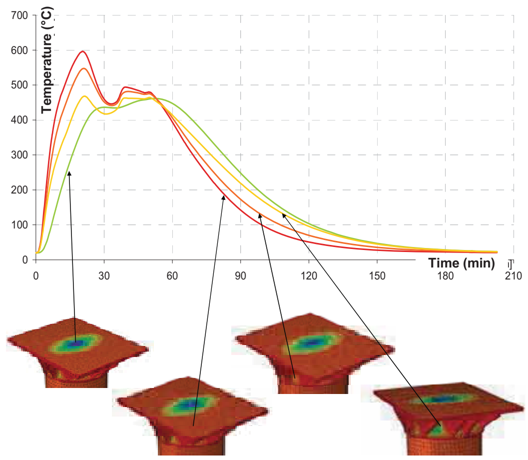



c) Time-displacement curves for the columns' top

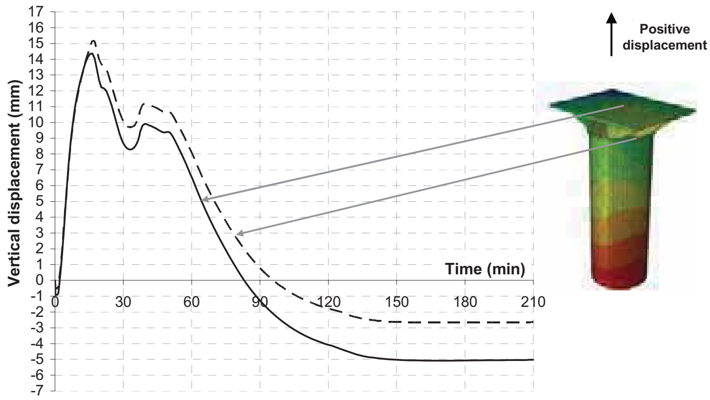



d) Stability check according to EN1993-1-2 (column 70)

Fig. 23.10 Scenario L2 – Global analyses




a) Columns time-temperature




b) Columns head time-displacement

Fig. 23.11 Scenario L2 – Detailed analysis.

## 23.3 SUMMARY

The Fire Safety Engineering approach thanks to advanced calculation models both for fire and for thermo-mechanical analysis of the structure, allows simulating the response behaviour of the structure exposed to “natural” fire scenarios. The FSE application to the car parks, to which this paper is dedicated, is allowed thanks to the information about the possible fire scenarios provided by the European Research Project CEC agreement 7215-PP/025 (2001). These fire scenarios may be of localised or generalised type as a function of the geometry and openings of compartment, namely of the ventilation conditions.

A natural fire is characterized by a heating phase and by a cooling phase. The thermal gradient in structural elements produced by the cooling phase is opposite to that produced by the heating phase. During the heating fire exposure there is a non-linear structural behaviour and plastic strains can be achieved in the structural members; for this reason, during the cooling phase the structure is different from the initial one. Therefore, after the cooling phase the stresses and the forces in the structural element can be different from the ones that could be found before the fire exposure.

The stresses and forces induced by constrained thermal deformations may cause structural collapse; however, they cannot be fully controlled by the prescriptive approach, as this approach is based on the assumption of a standard fire curve which increases unrealistically.

Finally, the thermo-mechanical analyses in fire situations for the described case study, consisting of the car parks located at the ground floor of buildings of the C.A.S.E. Project - L’Aquila, showed that the structures, and in particular the steel columns, considered unprotected, satisfy the performance level set to the design fire scenarios, also thanks to an overstrength in normal condition design.

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# 24 FIRE SAFETY DESIGN OF A MODULAR STEEL SYSTEM FOR HOTEL ROOMS

Summary

This paper presents a constructional solution developed for a modular steel structure, taking into account the fire safety design. This constructional solutionis intended to be used for construction of hotelrooms. According to the Portuguese Regulation, the minimum standard fire resistance of structural beams and columns is REI90, for the slab is REI 60 and for the partition elements is EI60 (Portaria nº1532/2008). Hot-rolled steel profiles are considered in the structural elements, and gypsum boards panels are used for the partition elements. The proposals for the partition elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data from manufactures of fire protection materials. Eurocodes 1 and 3 Part 1.2 (NP EN 1991-1-2:2010 and EN 1993-1-2:2005) are used for the design of the structural elements, taking into account the standard ISO 834fire curve.

## 24.1 GENERAL BUILDING DESCRIPTION

The study is based on a 4-floor hotel which isalready built using traditional construction in Vilamoura, Portugal. According to the Portuguese Regulation,this building is classified as type VII, 3$^{rd}$ category: Hotel and Catering lower than 28 m of high (DL nº220/2008). The unusual feature studied in this paper is the constructive solution for hotel buildings with prefabricated modules. The modules are made up with a steel structure (hot-rolled steel profiles) and partition elements. The evaluation of the fire safety performance of the structural solutions,developed (or proposed) by *OPWAY NovasTecnologias*, is the main topic of this study. Only one structural solution for the modular construction is detailed in this paper, taking into account the fire safety design. Two situations are studied: case A,the prefabricated module, for which columns and beams are respectively inserted in to walls and drop ceiling; case B, two modules are joined into a single one to make, for example, spacious rooms, and columns and beams are isolated. The solutions studied are summarized in Tab. 24.1. The proposals for the partition elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data of fire protection materials from manufactures. Tab. 24.2 shows the materials and their propertiesconsidered in this study.

Tab. 24.1 Summary of the studies

| | **Regulatory requirements** | **Case study** |
|---|---|---|
| **Walls** | EI60 | |
| **Drop ceilings** | EI60 | |
| **Columns** | REI90 | Case A: into walls (HEB 140) |
| | | Case B: isolated columns (HEB 140 + Promatec-H20 mm thick) |
| **Beams** | REI90 | Case A: into drop ceilings (HEA 140 + HEB 140) |
| | | Case B: isolated beam (HEA 140 + Knauf Fireboard 35 mm thick) |
| **Slabs** | REI60 | |

Tab. 24.2 Properties of the recommended protection materials

| | Plasterboard type *Knauf DF* | Plasterboard type *Knauf standard* | Plasterboard type *Knauf Fireboard* | Rock-wool | Calcium silicate board type *Promatec-H* |
|---|---|---|---|---|---|
| $l_p$ (W/mK) | 0.25 | 0.25 | 0.25 | 0.04* | 0.175 |
| $c_p$ (J/KgK) | 1700* | 1700* | 1200* | 840* | 1200* |
| $r_p$ (kg/m$^3$) | 750* | 750* | 800 | 40 | 870 |
| $p$ (%) | 20* | 20* | 20* | ≈ 0* | ≈ 0* |
| $e$ (mm) | 12.5 | 12.5 | 35 | 50/75 | 20 |

* Values tabulated in general bibliography; manufacturer's data do not provide this information.

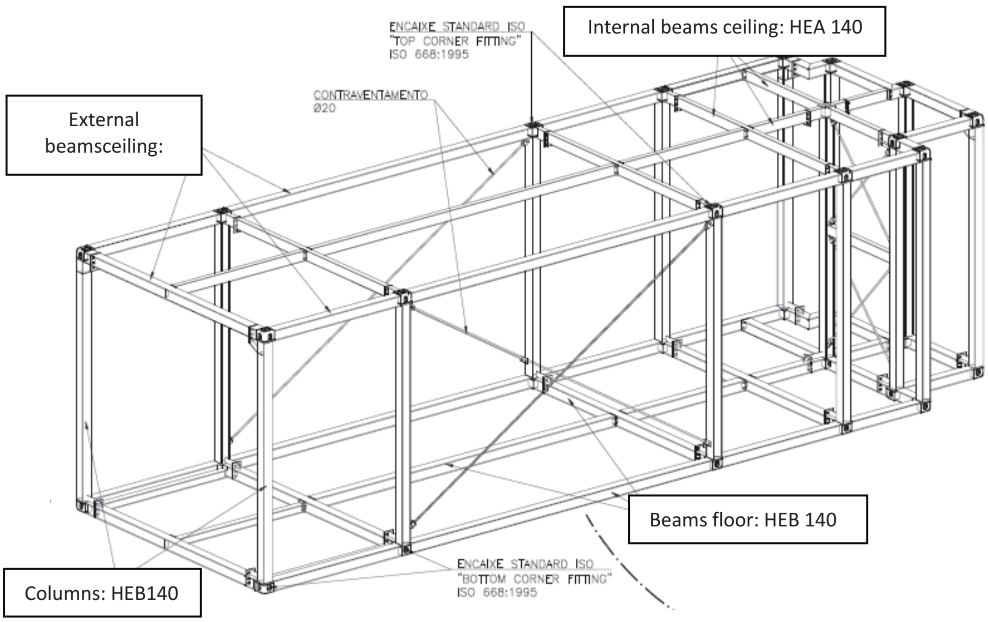



Fig. 24.1 Pre-fabricated module

## 24.2 ON WHAT PART OF THE PROJECT IS FIRE ENGINEERING USED AND THE PURPOSE OF CHOOSING A FIRE ENGINEERED APPROACH

Prefabricated modules are intended to be used for construction of hotel rooms, and the fire engineered approach is chosen in order to check the fire safety design of the modules.The design of structural elements (beams, columns and slabs) is performed according to Eurocodes 1 and 3, Part 1-2 (NP EN 1991-1-2:2010 and EN 1993-1-2:2005). The calculation procedure is based on a prescriptive approach and is carried out in the field of temperature.In many cases, the geometrical definition of the structural profiles insulationdid not fit to the typologies indicated in Eurocodes. Taking into account the safety of the structure, simplifications and approximations are performed to fit the problem in the Eurocodes typologies and formulations.

## 24.3 REGULATORY REQUIREMENTS FOR THE FIRE ENGINEERED PART OF THE PROJECT

### 24.3.1 Walls, ceilings and slabs

According to the Portuguese Regulation (Portarianº1532/2008), the building considered in this case study is part of thefollowing hazardous locations:

**D** - Establishment intended to receive children less than six years old or people with limited mobility or without ability of perception and response to an alarm (DL nº 220/2008, article 10);

**E**- Establishmentintended for overnight stays (DL nº 220/2008, article 10).

The minimum fire resistance of elements under ISO 834 standard fire curve for hazardous location D and E are given in articles 22 and 23 of Portarianº1532/2008 (Tab. 24.3). The most unfavourable situation for this study caseis toconsider thehazardous locationD.

Tab. 24.3 Hazardous locations for the analysed building

| **HAZARDOUS LOCATIONS E** | |
|---|---|
| **Construction elements** | **Minimum standard fire resistance** |
| Non-resistant walls | EI30 |
| Floors and resistant walls | REI30 |
| Doors | E15C |
| **HAZARDOUS LOCATIOND** | |
| **Construction elements** | **Minimum standard fire resistance** |
| Non-resistant walls | EI60 |
| Floors and resistant walls | REI60 |
| Doors | E30C |

Regarding the overall fire compartmentation, in general, the various levels must be defined as differentfire compartments. The maximum area of a fire compartment by floor for a type of use VII should be 1600m$^2$. These compartments must be isolated by building elements with a resistance class EI60 or REI60for type VIIuse, providing, at the minimum,spans with standard fire resistance class E30 (Portarianº 1532/2008, article 18).

### 24.3.2 Structural profiles (beams and columns)

According to the Portaria nº1532/2008, article 15, the minimum standard fire resistance of structural elements is given in function of the building category for different types of use. The present case study refers to a *type of use VII – accommodation and food services* (DL nº 220/2008, article 8) that corresponds to *buildings or parts of buildings receiving people, providing temporary accommodation or performing food services and drink activities, in exclusive occupation or not, namely those for tourism enterprises, local accommodation, food servicesor drinksestablishments, dormitories and, when not inserted in school establishment, student residences and holiday camps, campsites and caravanningbeing excluded and considered as spaces of use-typeIX*. The risk categories usedforuse-typeVII are presented in Tab. 24.4 (corresponds to Table VI of Annex III of DL nº 220/2008).

Tab. 24.4 Building categories for Use-type VII (UT)

| **Categories** | **Reference criteria** | | | **Hazardous location E with independent exits directly abroad in the reference plane** | **Study case (4-floors hotel)** |
|---|---|---|---|---|---|
| | **UT height** | **Number of persons of the UT** | | | |
| | | **Number of persons** | **Number of personsin hazardous location E** | | |
| 1st | ≤ 9m | ≤ 100 | ≤ 50 | Applicable to all | R30; REI30 |
| 2nd | ≤ 9m | ≤ 500 | ≤ 200 | Not applicable | R60; REI60 |
| **3rd** | **≤ 28m** | **≤ 1500** | **≤ 800** | **Not applicable** | **R90; REI90** |
| 4th | > 28m | > 1500 | > 800 | Not applicable | R120; REI120 |

## 24.4 ASSESSMENT STRATEGY: CONSTRUCTIONAL SOLUTIONS FOR THE PARTITION ELEMENTS

### 24.4.1 Interior and exterior walls

According to existing experience in this field, it is suggested that, to obtain a fire resistance of EI60, eachwall module should consist of two defined vertical elements(from outside to inside): i) 2 gypsum board panels (theexternal should be type *KnaufDF*and the internal type*Knauf standard*), with 12.5 mm thick each; ii)50 mm of low density rock-woolof 40 kg/m$^3$(except in the zone of the columns). In the zone of thecolumns,between the inner plate (type *Knauf standard)* and the columnsteel section, it should be placeda calcium silicate board, type*Promatec-H,*with 20 mm thick and 540 mm wide (corresponding tothe column width, with 200 mm moreat each side) as shown in Fig. 24.2. Thechange of the wallinsulation in the columns zone is related to the required fire resistance for columns (REI90), as explained in 24.6.2.

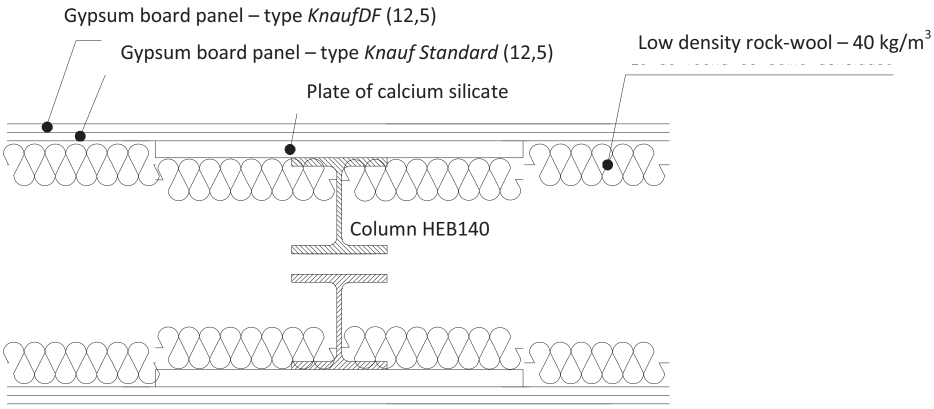



Fig. 24.2 Construction details of the wall between modules, plan view

The evaluation of fire safety requires the standardISO 834 fire from inside the building; therefore, in calculating the walls resistance, the more critical vertical element from the facade wallis the interiorvertical element. The solution for thefacade walls should be similar to the solution adopted for the inside walls: the interior vertical elementshould be equal to one of the vertical elements from the inside walls;for the exterior vertical elementthe indication of the thermal study shouldbe followed: i) 75 mm of low density rock-wool of 40 kg/m$^3$ (excluding the zone of the beams, in which it should be 50 mm); ii)*Aquapanel*board with 12.5 mm thick; iii) system of ventilated facades.

### 24.4.2 Drop ceilings

For the drop ceiling, it is recommended the application of (from bottom to top): i) 2plasterboards type*Knauf DF* with 12.5 mm thickness; ii) 50 mm of low density rock-wool of 40kg/m$^3$ (see Fig. 24. 3).

## 24.5 SELECTED DESIGN FIRE

The evaluation of fire safety requires the ISO 834 fire from inside the building.

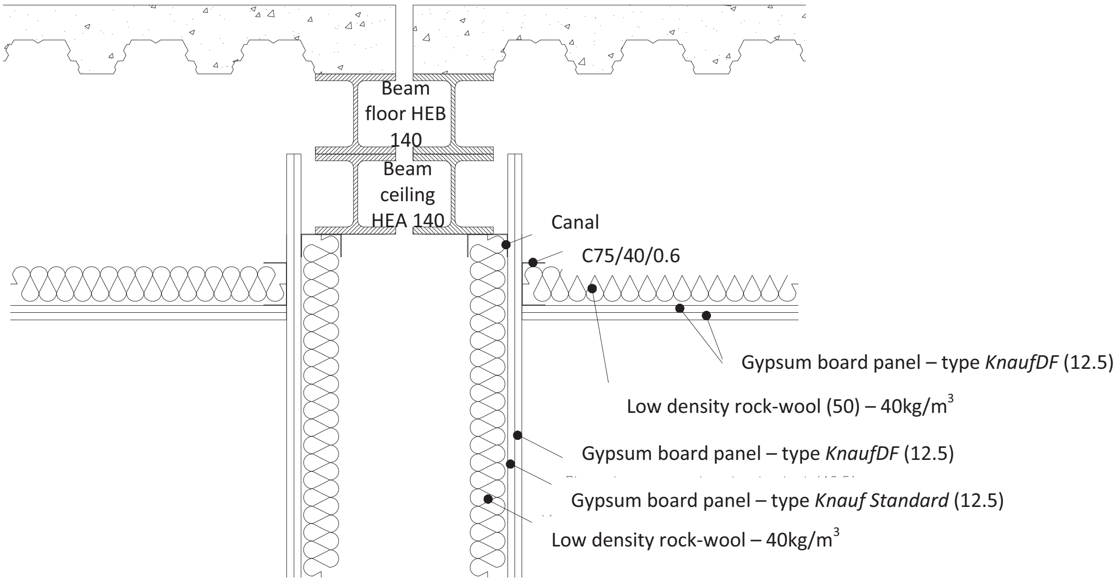



Fig. 24. 3 Construction details of the wall between modules, elevation view.

## 24.6 DESIGN OF STRUCTURAL ELEMENTS

### 24.6.1 Critical temperatures of the structural elements

The columns designis performed according to Eurocode 3, Part 1-2 (EN1993-1-2,2005),taking into account the ISO 834fire curve. The fire safety verification of the structureis made in the temperature field, imposing that the temperature does not exceed the critical temperatureduring the fire duration prescribed by the regulations (90 min.). In calculations, it is consideredthat the fire protection materials,used in the structural elements zone,sustain their insulationand resistance capabilities during 90 min. The temperature increase of an insulated steel memberduring the time interval $\Delta t$= 90 isgiven by:

$$\Delta\theta_{a.t} = \frac{\lambda_p / d_p}{c_a \rho_a} \frac{A_p}{V} \left( \frac{1}{1+\phi/3} \right) (\theta_{g.t} - \theta_{a.t}) \Delta t - \left( e^{\phi/10} - 1 \right) \Delta\theta_{g.t} \qquad (1)$$

wherethe amount of heat stored in the protection is given by:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V} \qquad (2)$$

where $c_p$ [J/kgK]is the specific heat of the fire protection material and $\rho_p$[kg/m$^3$]is the protection materialunit mass, $d_p$[m]is the thickness of the fire protection material; $c_a$[J/kgK]and $\rho_a$[kg/m$^3$]are thespecific heat and the unit mass of steel, and $A_p/V$ is the section factor for steel members insulated by fire protection material.

The critical temperature is calculated using the expression:

$$\theta_{cr} = 39.19 \ln\left(\frac{1}{0.9674 \mu_0^{3.833}} - 1\right) + 482 \tag{3}$$

where $\mu_0$ is the degree of utilization of the element at time t = 0, and $\mu_0$ can be obtained from:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} \tag{4}$$

where $R_{fi,d,0}$ is the value of $R_{fi,d,t}$ for time t = 0, for 20ºC, but calculated with the safety factors related tothe fire situation and $E_{fi,d}$ is the design effectof actions for the fire design situation, given by:

$$E_{fi,d} = \eta_{fi} E_d \tag{5}$$

where $E_d$ is the design effects of actions determined at ambient temperature, and $\eta_{fi}$ is the reduction factor for design load level in the fire situation. In this calculation, the basic live loadis considered to be the most representative value from the actions:

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} = \frac{1.0x(2.23 + 1.25 + 0.4x2) + 0.5 \times 2.0}{1.35x(2.23 + 1.25 + 0.4x2) + 1.5 \times 2.0} = 0.6 \tag{6}$$

The $E_{fi,d}$ values for each studied element are presented in the following tables (Tab. 24.5, and Tab. 24. 6).The ULS valuesare selected from the stability study at ambient temperature.

Tab. 24.5 Loads in the main columns at ambient and elevated temperature

| Column | | $N_{,Ed}$ (kN) | | $V_{z,Ed}$ (kN) | | $M_{y,Ed}$ (kNm) | | $V_{y,Ed}$ (kNm) | | $M_{z,Ed}$ (kNm) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Ext. 1 | Ext. 2 | Ext. 1 | Ext. 2 | Ext. 1 | Ext. 2 | Ext. 1 | Ext. 2 | Ext. 1 | Ext. 2 |
| HEB 140 | ULS | 363 | 363 | 6.7 | 6.7 | 20.1 | 2.1 | 1.9 | 1.9 | -4.3 | 2.3 |
| | fire | 218.3 | 218.3 | 4.0 | 4.0 | 12.1 | 1.3 | 1.1 | 1.1 | -2.6 | 1.4 |

Tab. 24.6 Loads in the main beams at ambient and elevated temperature

| | | $V_{z,Ed}$ (kN) | | $M_{y,Ed}$ (kNm) | | |
|---|---|---|---|---|---|---|
| | | Ext. 1 | Ext. 2 | Ext. 1 | Centre | Ext. 2 |
| Beam - floor | ULS | 39.4 | -38.6 | 0 | 78.1 | 0 |
| HEB140 | fire | 23.7 | -23.2 | 0 | 47.0 | 0 |
| Beam - ceiling | ULS | 16.5 | -16.5 | 0 | 21.3 | 0 |
| HEA140 | fire | 9.9 | -9.9 | 0 | 12.8 | 0 |

To define the critical temperatures, the following verifications are performed:

i) Columns – Buckling verification, and verification of the cross-section resistance under biaxial bending.

ii) Floor beams– Verification of the cross-section resistance to bending and axial forces. It is considered that the slab ensures the lateral restraint of the floor beams.

iii) Ceiling beams– Verification of the cross-section resistance to bending and axial forces. The lateral buckling is neglectedbecausethe loads are really small.

Tab. 24.7 presents the critical temperatures obtained for each studied element.In these calculations it is considered that all steel profiles are S275 class. Theconsidered effective length ofcolumns is $l_{fi}$ = 0.5 x 3.32 m (the maximum compression stressesoccur in the lower floors).

Tab. 24.7 Critical temperatures

| | Profile | Critical Temp. | Conditioning load |
|---|---|---|---|
| Column | HEB 140 | 621.9 ºC | Buckling by biaxial bending |
| Beam - floor | HEB 140 | 594.3 ºC | $M_{fi,Ed}$ |
| Beam - Ceiling | HEA 140 | 701.6 ºC | $M_{fi,Ed}$ |

**24.6.2 Columns**

The columns are defined by hot rolled HEB 140 profiles.Two situations are studied: case A, the column is inserted into walls, and it is considered that the fire protection comes from the materials constituting the walls (Fig. 24.1); case B, the column is not inserted into walls and is isolated with hollow encasement.

In case A, in order to calculate the temperature evolution in columns, fire acting only on one side of the cross section (fire inside the modules) is considered (Fig. 24. 4) and it is assumed that fire protection materials maintain their insulation and resistance capabilities for 90 min. It is also considered that the fire protection is ensured by the following materials: i) calcium silicate board, type *Promatec-H,* 20 mm thick and 540 mm wide; ii) 2 gypsum board panels (the external should be of type *KnaufDF*and the internal of type *Knauf standard*), with 12.5 mm thick each.

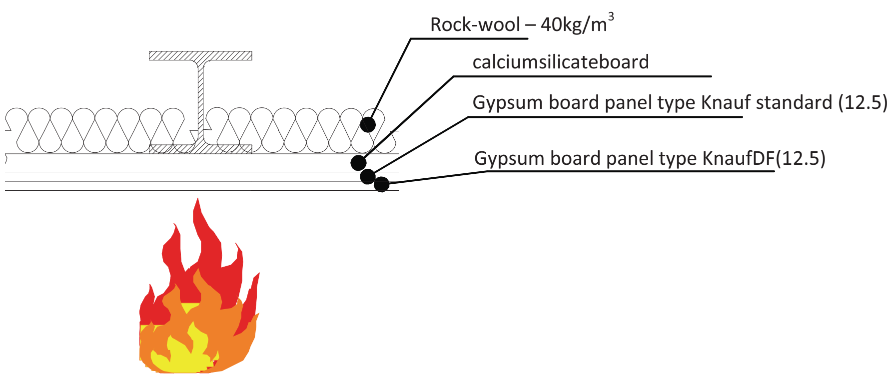



Fig. 24. 4 Representation of the fire action to the columns –Case A

The studied case B corresponds to twoisolatedprofilestogether, with hollow encasement, as showed in Fig. 24.5a). As this solution of two isolated profiles together is not directly considered in Eurocode3, Part1-2, the evolution oftemperature wascalculated for an approximatesituation: profileisolatedexposed tofireon all sideswith hollow encasement (Fig. 24.5b)). The fire protection is ensured bya calcium silicate board panel of type Promatec-H with 20 mm thick.

a) Twoisolatedprofilestogether b) Simplified situation

Fig. 24.5 Representation of the fire action to the columns –Case B

The section factor of the HEB 140 profile only exposed on one side is $A_p/V$ = 32.59m$^{-1}$, and exposed on four sides with hollow encasement is$A_p/V$ = 130.35m$^{-1}$. Fig. 24.6 shows the temperature evolution in the column cross section for the two cases, andthe steel temperature is lower than the critical temperature.The critical temperature of the HEB 140 profile is: 621.9°C and the steel temperature is: 111.5°C in case A, and 585.5°C in case B.

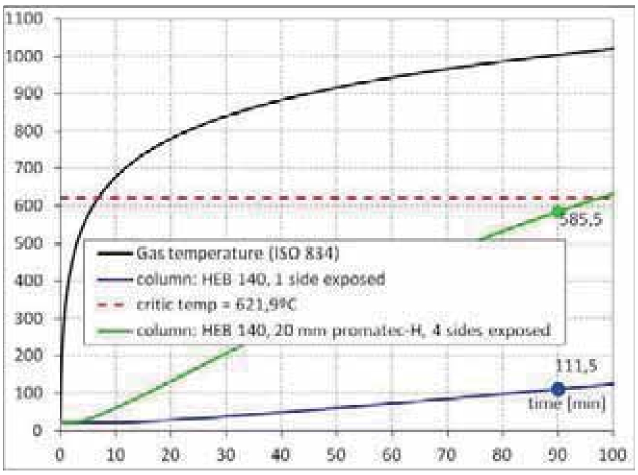



Fig. 24.6 Temperature evolution in steel column section during the ISO 834 fire

### 24.6.3 Interior and external beams

As for the column, the study also includesthe two study cases:case A, the beam is inserted into the ceiling; case B, the beam is out of the ceiling. The cross section of interior beams and external beams is the same, and is defined by hot rolled HEA 140 profile (ceiling beams) and HEB 140 profile (floor beams), as shown in Fig. 24. 3 (interior beams cross section).

For the case A, in order to design the beam section according to Eurocode 3, Part 1-2, the horizontal protection existing in the drop ceiling beneath the steel profiles is considered: i) 50 mm of low density rock-wool of 40kg/m$^3$; ii) 2 plasterboards *Knauf DF* with 12.5 mm thick (Fig. 24.7). Taking into account the position of the profiles, it is expected that the temperature of the floor beams is less than in the ceiling beams, which would lead to verify only this one; however, as the floor beams have a lower critical temperature, they are also verified in the same way: the horizontal protection existing in the drop ceiling beneath the floor beams is considered.The section factors of HEB 140 and HEA 140 profiles exposed on one side are, respectively, $A_p/V$= 32.59m$^{-1}$,$A_p/V$ = 44.59m$^{-1}$.

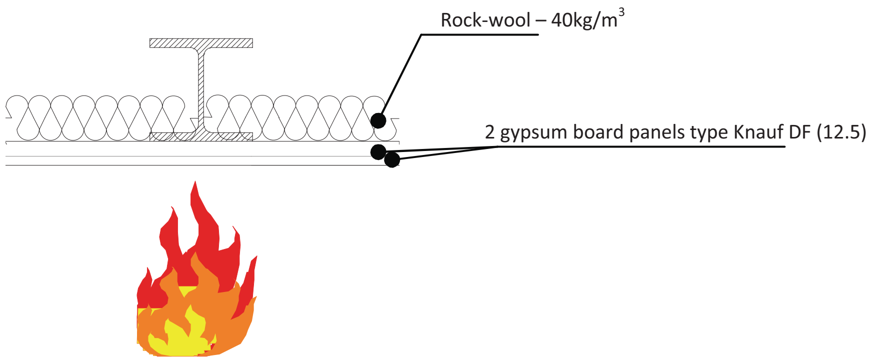



Fig. 24.7 Representation of the fire action to the beams –Case A

For the case B, it is considered that the beam resistance REI90 is ensured by the hollow encasement fire protection defined by a gypsum board panel type Fireboard, with 35 mm thick (Fig. 24.8). A few simplifications were also made to use Eurocode 3, Part 1-2: it was considered that the ceiling beams were exposed to fire on 3 sides, whereas the floor beams were not exposed to fire. The section factors of HEA 140 profiles exposed on three sides is $\mathrm{A_p/V}$= 129.22m$^{-1}$.

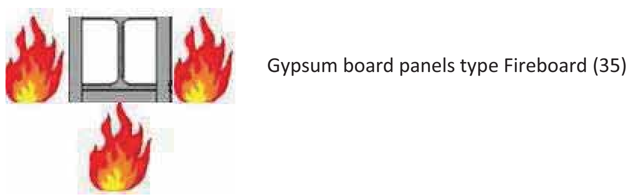
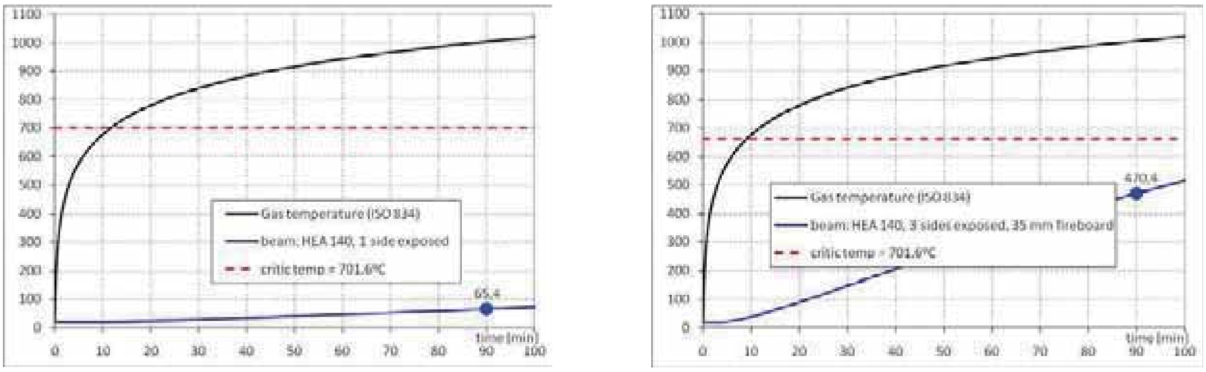



Fig. 24.8 Representation of the fire action to the beams HEA 140 – Case B

Fig. 24.9 shows the temperature evolution in beams HEA profiles for the two cases A and B. For both case, at the end of 90 min., the steel temperature is lower than the critical temperature.




a) Case A b) Case B

Fig. 24.9: Temperature evolution in steel beam section HEA 140 during the ISO 834 fire

#### 24.6.4 Slabs

The slab is "protected" by thedrop ceiling (Fig. 24. 3 – Slab cross section). The resistance of 60 min. (REI60) is guaranteed based on the materials insulation and resistance (EI60).

### 24.7 CONCLUSION

This paper presented the evaluation of a constructional solution developed for a modular steel structure, taking into account the fire safety design. Prefabricated modules are intended to be used for construction of hotel buildings. The results of this study are summarized in Tab. 24.8. The solution presented is in accordance withtherequired regulations, andhereafter are remembered the scope and limitations of this study:

i) The proposals for the partitions elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data of fire protection materialsfrom manufactures.

ii) The design of structural elements (beams, columns and slabs) is performed according to theEurocodes 1 and 3, Part 1-2. The calculation procedure is based on a prescriptiveapproach and is carried out in the field of temperature.

iii) In many cases, the geometrical definition of the structural profiles insulationdid not fit to the typologies indicated in Eurocodes. In these situations, simplifications and approximations,taking into account the safety of the structure, aremade to fit the problem in the Eurocodes typologies and formulations.

iv) The critical temperature is obtained by considering the steel grade S275, and the design effect of action for the fire situation is obtained from the design effect of action at ambient temperature and $\eta_{fi}$= 0,6 is used.

v) Use of advanced calculation modelscould lead to more economical solutions.

Tab. 24.8 Results summary

<table>
<tr><th></th><th>Regulatory requirements</th><th colspan="3">Study cases</th><th>$\theta_{cr}$ (ºC)</th><th>$\theta_{a,t}$ (ºC)</th></tr>
<tr><td>Walls</td><td>EI60</td><td colspan="5" rowspan="2">The proposals are based on the knowledge and experience of UC and the data of manufacturers of fire protection materials</td></tr>
<tr><td>Drop ceilings</td><td>EI60</td></tr>
<tr><td rowspan="2">Columns HEB 140</td><td rowspan="2">REI90</td><td colspan="2">Case A – columns inserted into walls</td><td></td><td>621.9</td><td>111.5</td></tr>
<tr><td colspan="2">Case B – isolated column</td><td>Promatec-H</td><td>621.9</td><td>585.1</td></tr>
<tr><td rowspan="3">Beams HEA 140 and HEB 140</td><td rowspan="3">REI90</td><td rowspan="2">Case A –beams inserted in ceiling</td><td>HEB 140</td><td rowspan="2">Knauf DF</td><td>594.3</td><td>54.3</td></tr>
<tr><td>HEA 140</td><td>701.6</td><td>65.4</td></tr>
<tr><td colspan="2">Case B – Isolated beams (HEA 140)</td><td>Knauf Fireboard (exposition 3 sides)</td><td>701.6</td><td>470.4</td></tr>
<tr><td>Slabs</td><td>REI60</td><td colspan="5">The 60 min. resistance must be ensured on the basis of the insulation and resistance of the drop ceiling insulation materials</td></tr>
</table>

References

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# 25 CASE STUDY: FIRE DESIGN VALIDATION

Summary

The case-study presents the particular validation of the advanced calculation model SAFIR (Franssen, 2005), asked by the Romanian authorities, for the verification of the fire resistance of the composite columns of a high-rise building.

## 25.1 INTRODUCTION

Using the computer program SAFIR, the authors made the verification of the fire resistance for the columns of "Bucharest Tower Center" high-rise office building, situated in Bucharest, Romania (Zaharia et al., 2007). The columns are made of partially concrete encased sections, with crossed I-sections, of octagonal and rectangular shape, as shown in Fig. 25.1.

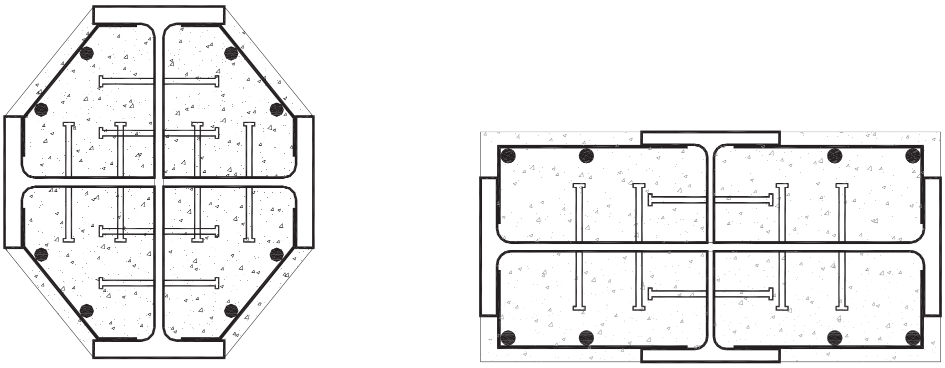

Fig. 25.1 Composite columns cross-sections

The fire resistance demand for these columns is of 150 minutes. The columns, considered as isolated elements and loaded with axial force and bending moments on both principal cross-axes (internal forces corresponding to the combination of actions for fire situation), were modelled with 3D beam elements. The buckling length of the columns was considered as the system length and equivalent imperfections according to the Eurocode specifications (EN 1994-1-1, 2005) were considered in both directions of the principal cross-section axes. The numerical analysis showed that the composite columns present a good behaviour under ISO fire; fire protection is needed only for a limited number of columns within the building, in order to attain the fire resistance of 150 minutes.

For this verification of fire resistance using an advanced calculation model, the Romanian authorities asked for a validation of the computer program SAFIR, using available experimental data of a fire test for a similar structural element. Partially concrete encased sections with crossed steel I-sections, used for the columns of “Bucharest Tower Center” building were promoted since 1987 by ARBED Luxembourg, in order to obtain high fire resistance times without supplementary fire protection (REFAO, 1987).

Therefore, the validation of the advanced calculation model SAFIR requested by the authorities considered a composite column with cross section similar to the cross sections of the “Bucharest Tower Center” columns, from the experimental report REFAO (1987), which presents some fire tests conducted on composite beams and columns made by steel profiles embedded in concrete.

## 25.2 FIRE TEST AND VALIDATION

The experimental specimen from the fire test considered for the validation, subjected to compression, is shown in Fig. 25.2 (REFAO, 1987). The test was conducted under standard ISO temperature-time curve. A fire resistance of 172 minutes was obtained, without supplementary protection for the profiles flanges.

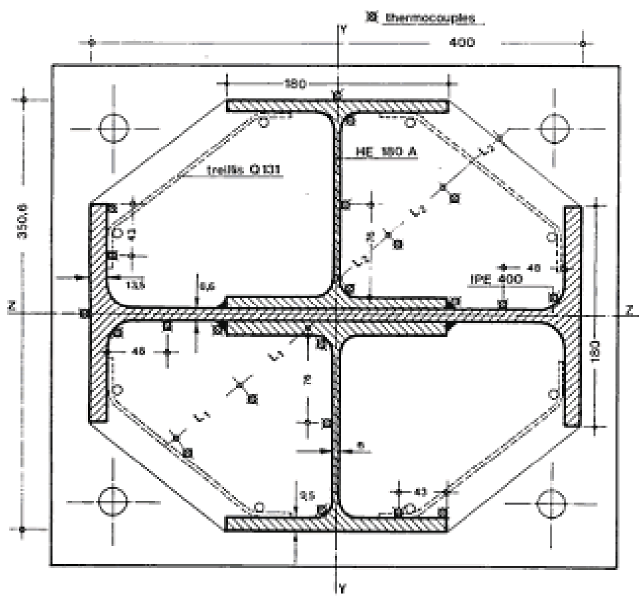


Fig. 25.2 Cross-section of the composite column tested under ISO fire (REFAO, 1987)

Fig. 25.3 shows the comparison between the measured temperatures within the composite cross section of the column (REFAO, 1987) and the temperatures obtained from the numerical analysis, at the failure time of the tested column of 172 minutes. It has to be mentioned that the resultant emissivities considered in the numerical model were the ones given in the test report for the surfaces of steel elements (0.3 and 0.5) and for the concrete (0.45). These values are lower than the surface emissivities given in EN1994-1-2 for steel and concrete (0.7), which were used for the fire resistance assessment of “Bucharest Tower Center” columns.

The numerical thermal analysis offers good results, with values which are close to the calculated temperatures, in the points from the cross section where the thermocouples were placed.

Considering a buckling length equal to the length of the column (both ends are pinned, as supposed to be in the test) the failure time given by the numerical analysis is of 132 minutes. This is a very conservative result, in comparison with the failure time of the experimental specimen of 172 minutes.

If a buckling length of 50% from column length is considered (both ends are fixed), the failure time obtained by numerical analysis is of 188 minutes, higher than the failure time obtained in the test. If a buckling length of 70% from column length is considered (intermediate situation between fixed and pinned supports), the failure time given by the numerical analysis is of 164 minutes. This suggests that for the tested column it was not possible to realize perfect pinned ends and there was a degree of rotational restraint at the supports.

The values of fire resistance times presented above were determined for a small initial global imperfection of 1 mm introduced in the numerical analysis, even if the test report states that no initial imperfection was emphasised after measurements.

## 25.3 CONCLUSIONS

SAFIR offers good results, in the safe side, in comparison with the results obtained from the fire test. Consequently, the advanced calculation model SAFIR was validated for the particular type of composite columns of “Bucharest Tower Center” building.

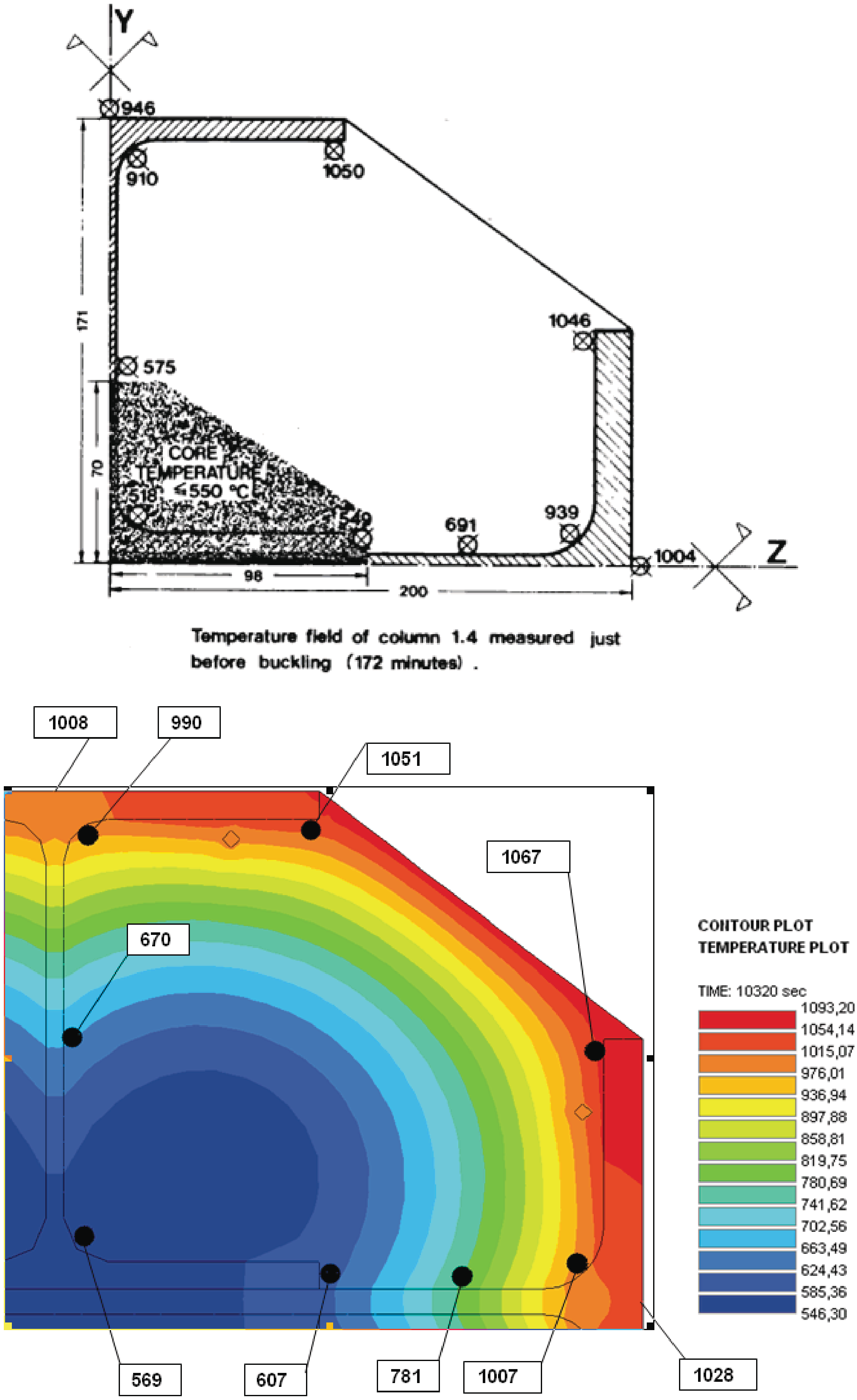


Fig. 25.3 Temperature distribution on cross-section:

experimental (REFAO, 1987) and numerical

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# 26 STRUCTURAL FIRE ANALYSIS: BLESSED TRINITY RC SCHOOL, BURNLEY

Summary

Fire engineering analysis has been carried out on a proposed three storey secondary school in order to demonstrate that the building will fulfill the functional requirement of the UK Building Regulations with regard to structural fire protection, with a reduced overall standard and coverage of said protection than that specified in the code. The principle outcome for the project contractor was decreased costs in terms of the amount of intumescent paint required and the associated costs of transportation and application. The analyses were carried out with the full consent of relevant stakeholders and approvals authorities following a detailed qualitative design review. The building is now complete, occupied and functioning as intended. The analysis consisted of determining the baseline equivalent fire resistance based upon likely fire severity in accordance with BS 7974 principles. A full frame finite element analysis was then undertaken to demonstrate that further fire protection could be removed from secondary structure without comprising the required structural stability.

## 26.1 SUMMARY

This chapter summarises in 17 pages the results of a fire engineering study which has been carried out to determine the level of structural fire resistance required to a sprinklered steel-composite framed building. It has been shown that a reduced 30 minute standard to main structural elements will fulfil the functional requirement of the UK Building Regulations without the need to protect much of the secondary structure given the characteristics of the design.

## 26.2 DESCRIPTION OF BUILDING

The building is a three-storey new build school in Burnley, Lancashire which will provide educational facilities for up to 1250 students. The building is to be constructed on a sloping site with the main entrance to the school located on Level 2 (upper ground floor). In addition to the general teaching areas at all levels, Level 1 provides dining and kitchen accommodation and a library, Level 2 has a theatre and stage area as well as a café and servery.

The building will have a sprinkler suppression system designed and installed in line with the current British Standard. The installation of the sprinkler system allows flexibility of design in terms of compartmentation and open spatial planning, facilitating the aspirations of the client and the Architect in terms functionality. The structure of the building is a three storey composite steel frame. Composite slabs are used in the building, 150mm thick reinforced concrete slab on multideck 50. All the beam-column connections and beam-beam connections are designed to be pinned joints.

### 26.3 REGULATORY REQUIRMENTS

The relevant regulatory requirements for the building are the UK Building Regulations. With regard to the fire protection of loadbearing elements of structure, the relevant Requirement B3 of the Building Regulations states, "The building shall be designed and constructed so that, in the event of a fire, its stability will be maintained for a reasonable period." To comply with Approved Document B (ADB), all loadbearing elements of structure should be provided with a minimum of 60 minutes fire resistance, given the height of the building and that the building will be fully sprinklered. Any structure which supports only the roof does not need to be fire resisting unless the structure is essential for the stability of an external wall which needs to have fire resistance. However the ADB recognises that alternative 'fire engineered' approaches may be used to achieve compliance with the Building Regulations. This provides an opportunity for a performance-based design approach.

### 26.4 FIRE ENGINEERED APPROACH

The fire engineered assessment of this building adopts a two-pronged strategy. First, the anticipated fire conditions within given enclosure are characterised with reference to a set time duration of the standardised gas temperature/time relationship described in BS 476 Part 20. The duration of exposure is derived empirically and referred to as the equivalent time of fire exposure or 'time equivalent' value. The time equivalency calculation method, described in PD 7974 Part 3, can be used to directly relate the severity of a real fire to an equivalent period of a standard fire resistance test, based upon the enclosure details, i.e. fire load, amount of ventilation, and thermal properties of the boundary materials. Safety factors are also incorporated into the calculation to take into account the likely risks and consequences of a fire. 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. Thus the baseline level of fire protection required for loadbearing elements of structure may be determined.

Following the time equivalency analysis a full frame finite element analysis is carried out with the baseline level of fire protection eliminated from secondary structural members. The intention is to show that the steelwork within each compartment (columns and beams with slabs above) will not fail when exposed to BS476 standard fire conditions for the minimum required fire resistance period. Vulcan software is used to carry out the finite element modelling.

## 26.5 PERFORMANCE/ASSESSMENT REQUIREMENTS

As the building is sprinklered throughout, it is assumed to be effectively compartmented on a floor by floor basis (i.e. a fire is unlikely to spread to more than one floor). Due to the high fire load in the library, it is proposed to enclose the library and its associated area in a separate compartment that achieves 60 minutes fire resistance. In addition to the library compartment, each floor of the building is divided into 5no compartments. Areas of special fire hazard will be enclosed in construction achieving 30 minutes fire resistance. Protected stairs will be enclosed in construction achieving at least 30 minutes fire resistance. It is assumed that a fire may involve a complete compartment, but will not spread to other compartments. Therefore it is reasonable to assess one fire compartment at a time in each phase of the analysis. Compartment layouts are shown in Fig. 26.1 – 26.3.

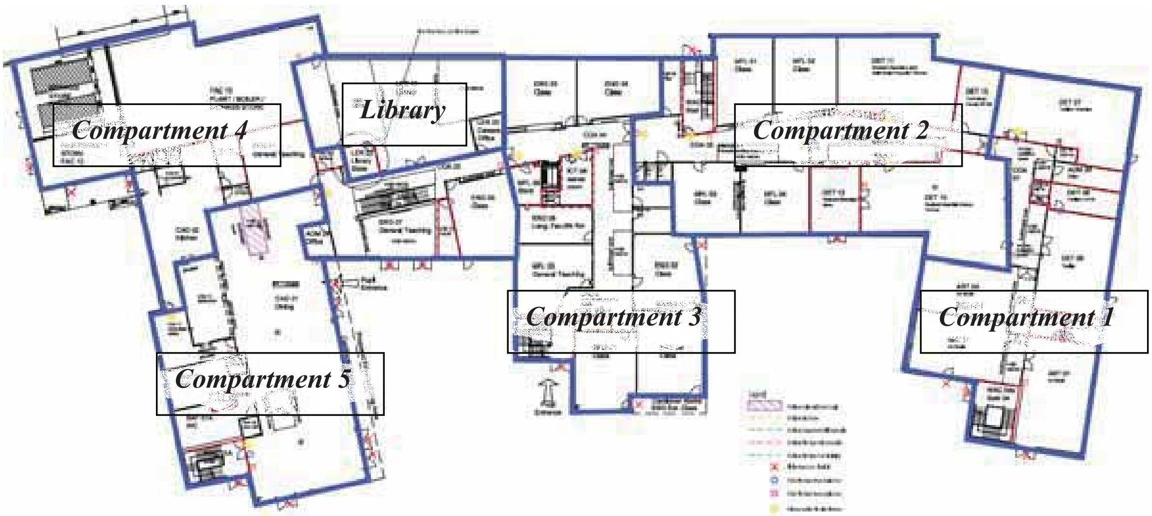


Fig. 26.1 Fire compartments on Level 1

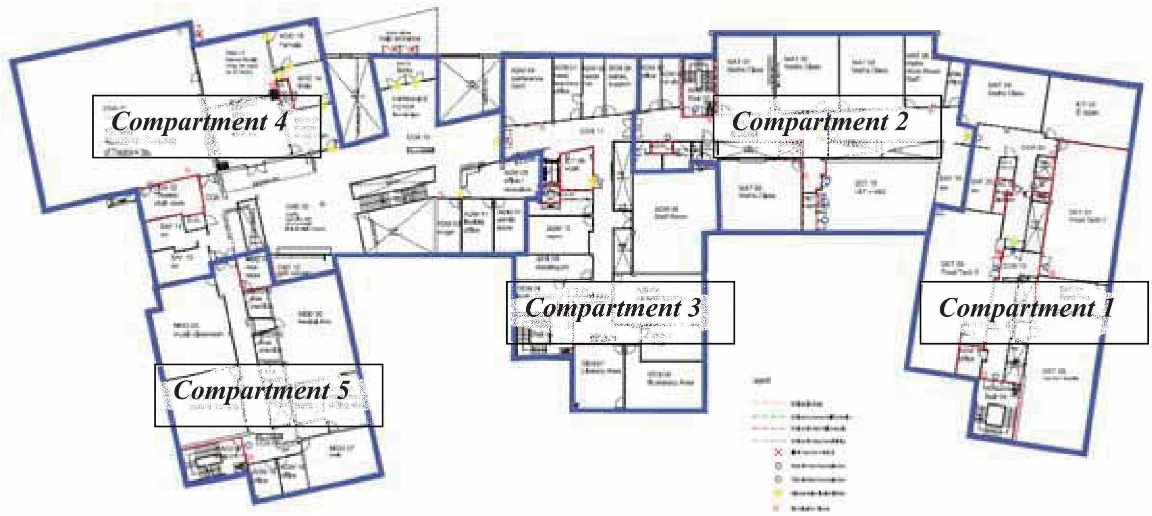


Fig. 26.2 Fire compartments on Level 2

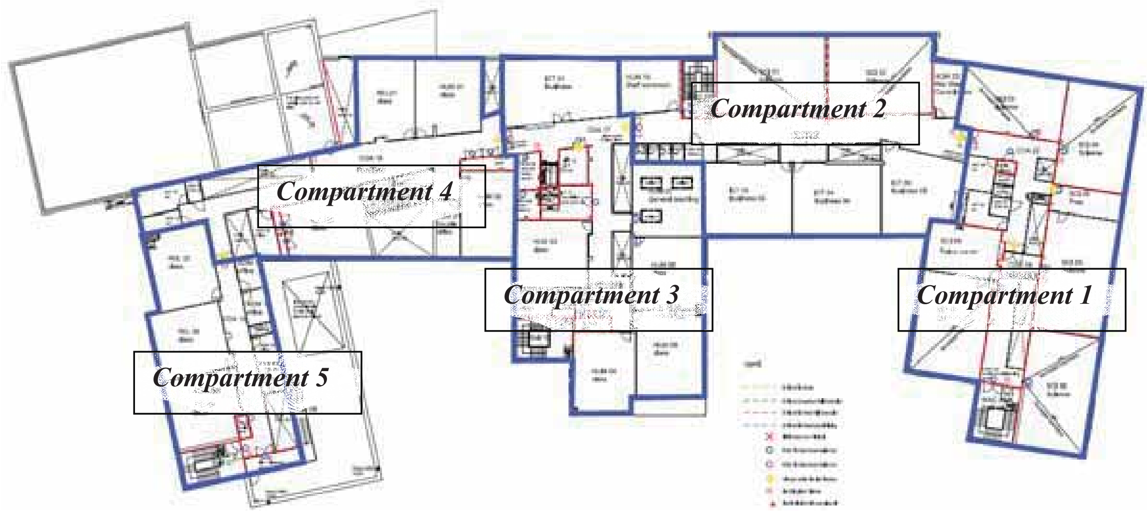


Fig. 26.3 Fire compartments on Level 3

### 26.5.1 Phase 1 Time Equivalence

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 sprinklers are assumed to have failed and the fire is assumed to burn until it runs out of fuel. Sprinkler intervention is 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. Fire brigade intervention is also 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.

### 26.5.2 Phase 2 Vulcan Analysis

Each sub-frame representing a compartment will be exposed to BS 476 standard fire for the required fire resistance period identified from Phase 1 or 30 minutes for places of special fire hazard. According to ADB, beams and columns should maintain the loadbearing capacity for the required fire resistance period when tested to BS 476. BS 476 Part 20 states that failure of a column to maintain its loadbearing capacity shall be deemed to have occurred when it fails to support the load. This can be checked by the finite element analysis as the finite element model stops once any one structural element loses its loadbearing capacity. The maximum deflection of a beam should be limited to $L/20$ where $L$ refers to the clear span. This limit is also applied to steel beams without external fire protection. ADB requires floors to maintain loadbearing capacity, integrity and insulation for the required fire resistance period. As for the beams, a slab shall be deemed to have failed if it is no longer able to support the load with a maximum allowed deflection of $L/20$ as required by BS 476. According to BS 476 Part 20, failure to maintain integrity shall be deemed to have occurred when collapse or sustained flaming on the unexposed face occurs or the criteria for impermeability are exceeded. Impermeability is normally satisfied for composite slabs. Should a slab collapse the finite element model will then stop. For composite slabs, insulation normally is not a problem.

Due to the nature of composite steel-framed buildings, localized failure (a limited number of structural elements losing stability) is unlikely to cause the whole frame to collapse. However, should the model stop before the required fire resistance period, it is considered that the structural frame does not meet the fire resistance requirements.

## 26.6 TIME EQUIVALENCE ANALYSIS

### 26.6.1 Fire Load

In order to determine the heat release rate of a fully involved compartment fire, it is necessary for the fire load within the compartments to be determined. PD 7974-1 provides statistical design data for typical fire load densities within various occupancies. However, it is noted in PD7974-1 that "the fire load densities assume perfect combustion, but in real fires, the heat of combustion is usually considerably less". Therefore the 80% fractile value is taken as the fire load density for design. This is the average value that is not exceeded in 80% of rooms or occupancies.

In accordance with Table A.19 in PD7974-1, the 80% fractile fire load density for school buildings is 360MJ/m$^2$; the 80% fractile fire load density for libraries is 2250 MJ/m$^2$. As the fire load density of libraries is much higher than normal school areas, it is recommended to enclose the library in a separate fire compartment to 60 minutes standard and apply 60 minute protection to elements of structure in this location.

### 26.6.2 Ventilation to the enclosure

Ventilation is accounted for in the time equivalency method by calculating a 'ventilation factor', $w_v$. This factor is based upon the size and orientation of the vents/openings, the floor area and the height of the enclosure and is taken from PD7974-3.

$$w_v = 1.7H^{-0.3}\left\{0.62 + 90\left(0.4 - {A_v}/{A_f}\right)^4\right\}\left(1 + b_v\, A_h/A_f\right)^{-1} \geq 0.5 \qquad (1)$$

$$b_v = 12.5\left\{1 + 10\left({A_v}/{A_f}\right) - \left({A_v}/{A_f}\right)^2\right\} \geq 10 \qquad (2)$$

$w_v$= ventilation factor

H = Height of the enclosure (measured from floor to ceiling)

$A_f$ = Floor Area

$A_h$ = Area of ventilation in the horizontal plane

$A_v$ = Area of ventilation in the vertical plane

$b_v$ = Calculation factor

When assessing the amount of ventilation to the enclosure, the following additional assumptions have been made:

- Testing has shown that the critical temperature for standard toughened glass is around 200°C. Therefore, it is generally assumed that all the glass without fire resistance will break shortly after fire ignition. However, a sensitivity study has been carried out to investigate the effect of glazing breakage. This analysis investigated both 75% and 100% of the glazing area of the enclosure breaking.
- The analysis is based upon perfect combustion for the duration of the fire, i.e. there is no growth period.

### 26.6.3 Thermal properties of the bounding materials

The heat loss to the structure and thermal properties of the bounding elements, e.g. walls, floors, ceiling, etc, are taken into account by the factor $k_b$. Values of $k_b$ are given in Tab. 26.4 of PD 7974-3 and are based upon the thermal inertia of the materials. The bounding materials used in this building are typical building surfaces, such as glazing, gypsum plaster and concrete which have a typical thermal inertia of between 720-2500 J/m$^2$s$^{1/2}$K. This gives a value of 0.07 for $k_b$.

### 26.6.4 Equivalent time of fire exposure, $t_e$

The equivalent time of standard fire exposure can be defined as:

$$t_e = k_b w_v q \tag{3}$$

$t_e$ = time equivalent value (min)

$k_b$ = factor relating to the thermal properties of the enclosure

$w_v$ = ventilation factor

q = fire load density (MJ/m$^2$)

### 26.6.5 Design Time Equivalent Duration, $t_{ed}$

While Equation 3 is a reasonable predictor of enclosure fire conditions, it does not consider those less quantifiable influences on the design fire duration. Therefore, various safety factors are added to take into account the building height, occupancy type and provisions of automatic sprinkler systems. The design time equivalent duration, $t_{ed}$ can be defined as:

$$t_{ed} = \gamma_1 \gamma_2 \gamma_3 t_e \tag{4}$$

$\gamma_1$ = safety factor reflecting the consequences of failure of the enclosure

$\gamma_2$ = safety factor reflecting the risk of a fully developed fire taking place

$\gamma_3$ = safety factor reflecting the benefits of installing active fire fighting measures

From PD7974-3, $\gamma_1$ is based upon the height of the enclosure where for an assembly building less than 20m high, $\gamma_1$ has a value of 0.8. PD7974-3 specifies that safety factor $\gamma_2$ for assembly buildings has a value of 0.8 and recommends safety factor $\gamma_3$ of 0.6 where an automatic sprinkler system is provided.

**26.6.6 Design Time Equivalence Results**

The design time equivalence results are tabulated in Table 1 (compartments designated DET represent small 30 minute rooms of special fire risk). The results show that the worst case is fire compartment 1 on Level 3 which gives a design time equivalent duration of 27.5 minutes. The classrooms in this compartment are all enclosed in 30 minutes fire rated enclosure. Therefore only the corridor and WCs are included in the floor area and there is no ventilation to this area. This is onerous as the fire load in corridor and WCs is considerably low.

The results shown in Tab. 26.1 are considered onerous given that:

- The analysis is based upon the maximum burning rate for the duration of the fire, which does not take into account the growth stage of the fire;
- The fire is assumed to be under perfect combustion. In reality, this would be unlikely and the compartment temperatures would be less;
- In the fires that involve more than one room in the case of cellular plan layout, the time taken to burn through the dividing wall has not been taken into account. Although they have no defined fire resistance, there will be an inherent delay in fire spread;
- Areas such as corridors and WCs with very low fire risk are assumed to have the same fire load density;
- Automatic fire suppression systems will control the growth and spread of a fire. Accordingly, fires can be controlled within an area of burning consistent with the spatial configuration of the suppression system.

Therefore any fire will likely be contained within a small area of fire origin due to the combination of walls, floors and sprinklers. It is very unlikely that the whole compartment would be involved in the fire. In addition, assuming the sprinkler system achieves its intended function of controlling the fire size, the temperatures within the compartment can generally be assumed to be limited to 1000°C (unprotected steel maintains its strength up to 400°C).

The analysis shows that 30 minutes fire resistance to elements of structure would be sufficient, except 60 minutes for the library compartment. The walls and floors separating different compartments should also have 30 minutes fire resistance, except 60 minutes for the ones enclosing the library.

Tab. 26.1 Design Time Equivalence Results

| Fire Scenario | Required Fire Resistance Period (mins) |
|---|---|
| Level 2 Compartment 1 | 16.29 |
| DET03 | 30 |
| DET03 01 & 04 | 30 |
| Level 2 Compartment 2 | 15.76 |
| DET15 | 30 |
| Level 2 Compartment 3 | 15.9 |
| Level 2 Compartment 4 | 21.86 |
| Level 2 Compartment 5 | 19.3 |
| Level 1 Compartment 1 | 12.67 |
| DET07 | 30 |
| Level 1 Compartment 2 | 18.86 |
| DET11 | 30 |
| DET10 | 30 |
| Level 1 Compartment 3 | 14.75 |
| Level 1 Compartment 4 | 15.65 |
| Plant Room | 30 |
| Kitchen | 30 |
| Level 1 Compartment 5 | 12.46 |

## 26.7 FINITE ELEMENT ANALYSIS VULCAN

### 26.7.1 Software

Vulcan is a finite element analysis (FE) program capable of modelling the global 3-dimensional behaviour of composite steel-framed buildings under fire conditions. The analysis includes geometrical and material non-linearity with full membrane action in the slabs and cracking in concrete. Standard stress-strain curves and full thermal expansion characteristics are incorporated as functions of temperature for both steel and concrete, with uniform or non-uniform temperature distributions. The orthotropic nature of composite deck slabs is represented using an effective-stiffness concept. Vulcan output has been extensively validated against a range of data including test results of fire test programmes at the BRE Cardington facility.

The structure is modelled as an assembly of finite beam-column, spring, shear connector and slab elements. It is assumed that the nodes of these different types of element are defined in a common fixed reference plane, which coincides with the mid-surface of the concrete slab element. The beams/columns are represented by 3-noded line elements. The cross-section of each element is divided into a number of segments to allow variations in the temperature, stress and strain through the cross-section to be represented. Both geometric and material non-linearities are included. To represent steel-to-steel connections in a frame, a 2-noded spring element of zero length, with the same nodal degrees of freedom as a beam-column element, is used. The interaction of steel beams and concrete slabs within a composite floor is represented using a linking two-noded shear-connector element of zero length, with three translational and two rotational degrees of freedom at each node. The analysis includes geometric non-linearity in the slabs, using a quadrilateral 9-noded higher-order isoparametric element using an effective stiffness model to model the ribbed nature of typical composite slabs. The temperature and temperature dependent material properties can be specified independently. A maximum-strain failure criterion has been adopted for the concrete, and a smeared model has been used in calculating element properties after cracking or crushing. After the initiation of cracking in a single direction, concrete is treated as an orthotropic material with principal axes parallel and perpendicular to the cracking direction. Upon further loading of singly cracked concrete, if the tensile strain in the direction parallel to the first set of smeared cracks is greater than the maximum tensile strain then a second set of cracks forms. After compressive crushing, concrete is assumed to lose all stiffness. The uniaxial properties of concrete and reinforcing steel at elevated temperatures, specified in Eurocode 4: Part 1-2 have been adopted in this software.

### 26.7.2 Membrane Action

Membrane action of concrete floor slabs is due to the development of in-plane forces within the depth of the slab. Depending on the horizontal restraint conditions around the slab's perimeter, membrane action can occur at small and large vertical displacements. During a fire large displacements of floor slabs within a building are acceptable provided structural collapse or compartmentation failure does not occur. These large displacements lead to tensile membrane action occurring within the slab, which can be beneficial to the survival of the building. For slabs that have no horizontal restraint around their perimeter, membrane action can still develop provided the slab is two-way spanning.

### 26.7.3 Finite Element Models

The purpose of this analysis is to demonstrate that each fire compartment (sub-frame) meets the required fire resistance period identified in Phase 1. Each compartment will be modelled separately. In order to simulate the interaction between heated structure and cool structure and to model the loads accurately, a bay of sub-frame within the compartments adjacent to the fire compartment is also included in the model. All the columns above and under the compartment floor are included.

### 26.7.4 Material Properties

All of the steelwork in the building is grade S355. The concrete used is C28/35 (minimum) with characteristic cube strength of 35N/mm$^2$. Cold worked reinforcement bars with a characteristic strength of

460N/mm$^2$ are used in the slabs. The tensile strength of concrete is assumed to be zero. The temperature dependent material properties are taken into account in the finite element models.

### 26.7.5 Modelling slabs

The composite slabs used in the building are constructed on steel decking multideck 50-V2 with a total depth of 150mm and a layer of minimum A142 mesh in top (a total of 142mm$^2$ of mesh per metre length of slab in each direction, with 25mm cover). Full composite action between the beams and slabs is assumed. To be conservative, only the continuous depth of the slabs has been considered being 100mm thick. Ignoring the bottom 50mm of the slabs results in higher temperature, lower strength and therefore higher deflection. The contribution of the steel decking is also ignored. Instead of using a discrete series of bars, Vulcan considers the mesh as consisting of two smeared layers, reaching across the length of the slab and spanning in opposite directions. To conform to the properties of the 'true' mesh, the thickness of each layer is calculated such that they have an area of 142mm$^2$/m (i.e. each layer is 0.142mm thick). The slab cross-section is divided into 10 layers. Layering is important as it allows the temperature distribution within the slab to be modelled accurately.

### 26.7.6 Connections

All the beam-column connections and beam-beam connections are designed to be pinned joints. Spring elements have been used to simulate these connections. The springs are assumed to have indefinite axial stiffness but are free to rotate.

### 26.7.7 Effects Of Fire

The effects of fire on the structure are included in the models by defining the temperature profile of structural elements. The elements in the fire compartment are assumed to be heated under BS 476 standard furnace fire condition while the elements outside the fire compartment remain ambient temperature (20°C). Two aspects of temperature need to be defined: the time-temperature relationship, and the temperature profile through individual sections. For the analysis the steel frame in each compartment is initially modelled without any fire protection, except the main beams and columns that fall within 30 minute rooms which are protected to 30 minutes standard. If the model stops due to localized failure (a limited number of structural elements lose stability) at a lower time than the time equivalency then certain columns and/or beams are protected and the model is re-run. This procedure is repeated until the model has sufficient protection to comfortably exceed the time equivalency value. The selection of which columns and/or beams are to be protected is made by inspecting the behaviour of the models.

### 26.7.8 Thermal Response of Structural Steelwork

The performance of structural elements in fire depends upon the way in which they are heated, on the temperatures reached, on the materials used and on the way they are stressed. The method of determining the thermal response of the structural steel members is described in PD 7974-3. Uniform temperature distribution over the cross-section is assumed for steel beams and columns. For an equivalent uniform temperature distribution in the cross-section, the increase of temperature ΔT in an unprotected steel member during a time interval Δt should be determined from:

$$\Delta T = \frac{H_p / A}{c_a \rho_a} q_{net} \Delta t \tag{5}$$

Hp/A is the section factor for an unprotected steel member (m-1),

$q_{net}$ is the net heat flux (W/m$^2$),

$c_a$ is the specific heat of steel (J/kgK),

$\rho_a$ is the unit mass of steel (kg/m$^3$),

For an equivalent uniform temperature distribution in the cross-section, the increase of temperature ΔT in a protected steel member during a time interval should be determined from:

$$\Delta T = \frac{\lambda_i}{d_i} \frac{H_p / A}{c_a \rho_a} \frac{(T_g - T_a)}{(1 + \Phi / 3)} - (\exp(\Phi / 10) - 1)\Delta T_g \tag{6}$$

with

$$\Phi = \left( \frac{c_i \rho_i}{c_a \rho_a} \right) d_i \left( \frac{H_p}{A} \right) \tag{7}$$

where

$T_g$ is the flame temperature (°C),

$T_a$ is the steel temperature (°C),

$\lambda_i$ is the thermal conductivity of the fire protection material (W/mK),

$d_i$ is the thickness of the fire protection material (m),

$c_i$ is the specific heat of the fire protection material (J/kgK),

$\rho_i$ is the unit mass of the fire protection material (kg/m$^3$).

The steel temperature at time t is determined from the steel temperature at the previous time step, plus the temperature increase ΔT during the previous time increment. Steel beams are assumed to expose to fire on three sides while columns on all four sides. Protected beams and columns have been assumed to reach 550°C at 30 minutes.

#### 26.7.9 Thermal Response of Floor Slabs

Slabs are considered to be heated only on the bottom. Slab temperatures are calculated using the built-in function of Vulcan (Eurocode 4: Part 1.2 method). As the bottom 50mm of the slab has been ignored in the models, the calculated temperatures are relatively higher than they should be.

#### 26.7.10 Boundary Conditions

Boundary conditions are applied to the top and bottom of the columns only. The top of the columns is permitted to move only in a vertical direction so that superstructure loading may be applied. The bases of the columns are assumed to be completely fixed in place.

#### 26.7.11 Loading

The structural effects of a fire in a building, or part of a building, should be considered as a fire limit state. A fire limit state should be treated as an accidental limit state. In checking the strength and stability of the structure at the fire limit state, the loads should be multiplied by the relevant load factors. These are given in BS 5950-8. Loadings were supplied by the structural engineers.

### 26.8 RESULTS OF FULL-FRAME ANALYSIS

The finite element model stops once any one structural element loses stability. Although localized failure (a limited number of structural elements lose stability) is unlikely to cause the whole frame to collapse, it is considered that the structural frame loses stability at the time the model stops. Tab. 26.2 compares the analysis results with the required fire resistance period for each fire compartment. It can be seen that given the defined fire protection every compartment maintains the loadbearing capacity for the required fire resistance period with additional safety margin. An indicative mark-up of the fire protection required to the steelwork in one area is shown in Fig. 26.6. It may be seen that the library structure remains as 60 minutes, but much of the secondary protection elsewhere may be eliminated.

As a result of the finite element modelling, the floor deflections can be output as graphed for example in Fig. 26.4. It can be seen from the figures that the maximum floor deflection in each compartment is less than $L/20$ at the end of the analysis except for Level 1 compartment 2. In Level 1 compartment 2 the $L/20$ limit is reached at 28 minutes while the required fire resistance period for this compartment is only 18.86 minutes. Therefore, the loadbearing capacity and integrity of the whole structure is maintained when exposed to a foreseeable fire. ADB states that compartment walls should be able to accommodate the predicted deflection of the floor above by either: a) having a suitable head detail between the wall and the floor, that can deform but maintain integrity when exposed to fire; or b) the wall may be designed to resist the additional vertical load from the floor above as it sags under fire conditions and thus maintain integrity.

Tab. 26.2 Summary of finite element modelling results

| Fire Scenario | Required Fire Resistance Period (mins) | FE analysis results |
|---|---|---|
| Level 2 Compartment 1 | 16.29 | Model stopped at 20.16 minutes |
| DET03 | 30 | Model ran for 30 minutes without failure |
| DET03 01 & 04 | 30 | Model ran for 30 minutes without failure |
| Level 2 Compartment 2 | 15.76 | Model stopped at 19.81 minutes |
| DET15 | 30 | Model ran for 30 minutes without failure |
| Level 2 Compartment 3 | 15.9 | Model stopped at 18.08 minutes |
| Level 2 Compartment 4 | 21.86 | Model stopped at 25.89 minutes |
| Level 2 Compartment 5 | 19.3 | Model stopped at 20.91 minutes |
| Level 1 Compartment 1 | 12.67 | Model stopped at 15.53 minutes |
| DET07 | 30 | Model ran for 30 minutes without failure |
| Level 1 Compartment 2 | 18.86 | Model ran for 30 minutes without failure |
| DET11 | 30 | Model ran for 30 minutes without failure |
| DET10 | 30 | Model ran for 30 minutes without failure |
| Level 1 Compartment 3 | 14.75 | Model stopped at 16.05 minutes |
| Level 1 Compartment 4 | 15.65 | Model stopped at 16.81 minutes |
| Plant Room | 30 | Model ran for 30 minutes without failure |
| Kitchen | 30 | Model ran for 30 minutes without failure |
| Level 1 Compartment 5 | 12.46 | Model stopped at 14.59 minutes |

### 26.8.1 Connections

Although connections are critical members in structures, it is only now that they are receiving enough attention to achieve better understanding of their behaviour in fire. The reason of not requiring assessment of connection behaviour in fire in design is based on the argument that in the connection region, the fire exposed area is low compared with the mass, thereby slowing down temperature rises in the connection region in fire compared with the connected beams and columns. However, recent observations from real fires show that, on some occasions, the accumulative effects of a number of factors (including hogging bending moment, tension field action in shear and high cooling strain or pulling in effect at large deflections of the connected beam) could make the tension components of the connections fracture. In general, steel beams restrained by the adjacent structure would go through a number of phases in behaviour when exposed to fire, including combined flexural bending and compression during the early stage of fire exposure, as a result of restrained thermal expansion of the beam, through flexural bending at temperatures around the beam's limiting temperature under pure bending, and finally to catenary action at the late stage of fire exposure when the beam deflection is very large. The compression forces in the connections are caused by restrained thermal expansion of the beams while the tension forces are caused mainly by the contraction of the beam under large deflection. The connection spring elements are assumed to have indefinite axial stiffness but free to rotate. Fig. 26.5 shows the axial forces in typical connections during heating extracted from the finite element models. By assuming indefinite axial stiffness in the connections, the deformations of the connections are neglected. When exposed to fire, steel gradually loses stiffness and strength. As such at elevated temperature the axial stiffness of the connections will also reduce which allows the beams to expand and contract to some extent. As a result the axial forces in the connections will be lower than the calculated. Nevertheless the calculated results still indicate the trend of the connection forces. It can be seen from Fig. 26.5 that the connections are still under compression at the end of the fire exposure. Therefore connection fracture is unlikely to occur.

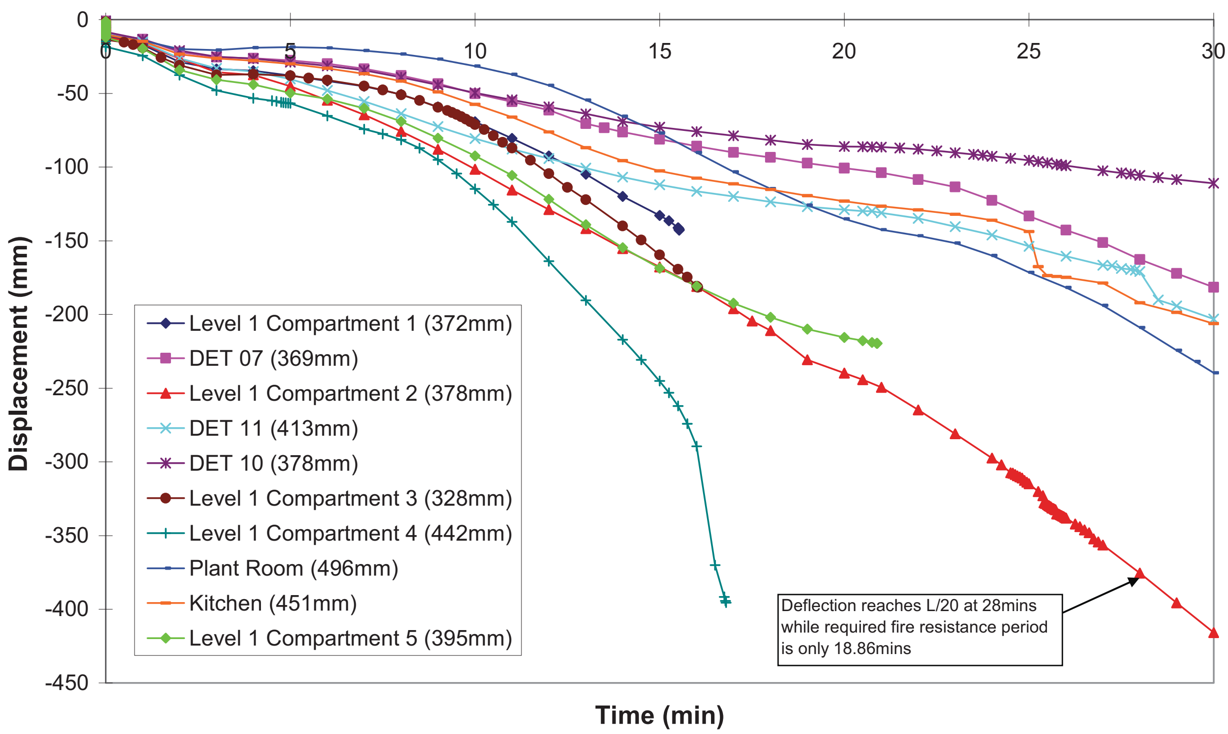


Fig. 26.4 Maximum deflection of Level 2 slabs while exposed to fires on Level 1

(max. allowable deflection L/20 indicated in brackets)

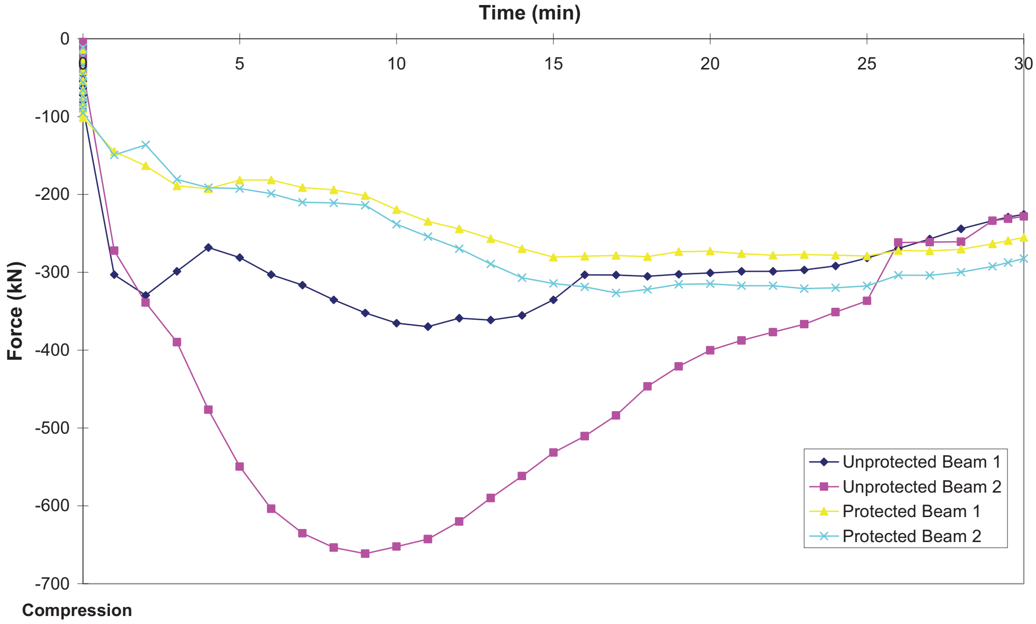


Fig. 26.5 Axial forces in typical connections

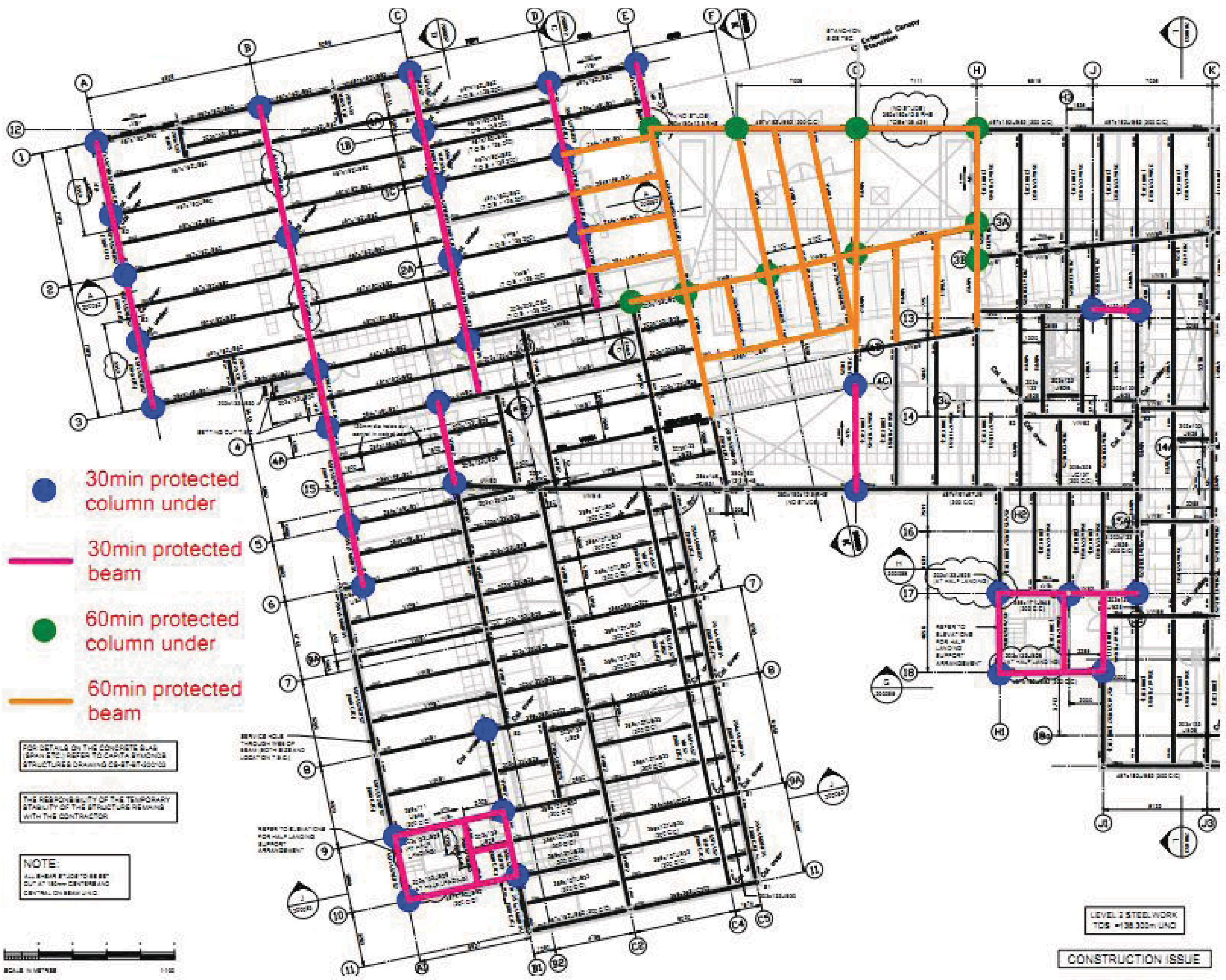


Fig. 26.6 Fire protection steelwork mark-up Level 2 (partial)

## 26.9 STAKEHOLDER APPROVAL

Before undertaking the fire engineered analyses it was necessary to hold meetings with the relevant stakeholders (Contractor, Building Control and Fire Service) both to explain the principles and intent and also to agree certain study parameters such as ventilation sensitivity studies for the time equivalence analysis. This method of gaining stakeholder approval prior to performing the analysis is in concurrence with the Qualitative Design Review (QDR) procedure as defined in BS 7974.

Due to Exova Warringtonfire's extensive experience in detailed fire engineered analysis no significant problems were encountered when seeking approval for the analysis. Questions received generally related to the ongoing maintenance requirements for the project once complete.

## 26.10 CONSEQUENCE ON LIFECYCLE OF BUILDING

The principle consequence on the lifecycle of the building relates to future use should a subsequent owner wish to change the principle compartmentation lines on a given floor. The recommendations are also contingent on a sprinkler system being maintained on the premises.

Additional important points for the contractor were as follows:

- Reinforcement mesh in the slab must be sufficiently lapped to form a full tension lap.
- All beams should be composite and the slabs should be tied to the beams.
- Connections to protected columns or beams should also be protected.
- Connections should be designed to be ductile.

As the use of the building will be a secondary school it was considered that the lifecycle of the building within this use group would be extensive and therefore compartmentation lines are unlikely to change during this period. However, the relevant stakeholders were made fully aware of this issue prior to undertaking the analysis and informed that any future change of use may be restricted should the current structural fire protection scheme wish to be maintained.

## 26.11 CONCLUSIONS

Fire engineering analysis has been carried out on the described building in order to demonstrate that the building will fulfill the functional requirement of the UK Building Regulations with regard to structural fire protection, with a reduced overall standard and coverage of said protection. The principle outcome for the project contractor was decreased costs in terms of the amount of intumescent paint required and the associated costs of transportation and application. The reduction in required fire protection is significant and may indicatively be seen in Figure 6 for one area of the building. The analyses were carried out with the full consent of relevant stakeholders and approvals authorities following a detailed QDR. The building is now complete, occupied and functioning as intended.

Acknowledgement

The authors would like to thank Bovis Lend Lease Ltd, Ellis Williams Architects, Carillion Plc, Lancashire Fire and Rescue Service and Andrew McCracken of Exova Warringtonfire for their contribution and assistance throughout the project.

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# 27 STRUCTURAL FIRE ASSESSMENT OF THE ME HOTEL ALDWYCH, LONDON

Summary

This case study outlines a structural fire engineering review that has been carried out on the atrium steelwork within the ME Hotel project in London. Here a methodology to determine the amount of fire protection on the atrium steelwork is described, whilst meeting the requirements of the Part B of the English Building Regulations. In principle, the assessment was done in two steps; a hazard identification and risk assessment, and a structural analysis at elevated temperatures. For the worst fire scenario a heat transfer and thermal structural finite element analysis was carried out. An initial fire protection regime was proposed for the finite element study and is subsequently proven to be adequate in maintaining the global stability of structure. At ground floor level, where the frame will enclose a single storey compartment, the steelworks will be provided with 120-minute fire protection. From the first to tenth floor the steel frame encloses the atrium; the steelwork at the first floor level and the corner elements that run up the apexes will be protected to a 60-minute standard. The remainder of the atrium structure can be left unprotected.

## 27.1 INTRODUCTION

This case study describes the structural fire engineering assessment conducted on the ME Hotel project in London. The building consists of a mixed hotel-residential development. The residential apartments are located in the North East end of the building whereas the hotel accommodation is located in the remainder of the building. See Fig. 27.1 and Fig. 27.2 for plan and elevation of the building. In this case study only the hotel part of the building is looked into.

The main feature of the building is a steel truss structure, which is continuous from the Ground Floor to the uppermost hotel floor. Being triangular in plan, this structural frame gradually decreases in area with respect to height. The enclosed area within the triangular structure at ground floor level would be a restaurant; the structure from first floor onwards would be an open atrium space, to be used as a reception and check-in area. The atrium structural frame will be clad with a natural marble; this cladding forms a complete physical separation between the atrium and the hotel corridors which surround it.

This atrium is structurally crucial to the building by:

- Carrying gravity loads – Secondary beams carrying the floor slabs frame into the edge beams which form part of the atrium structure.

- Providing lateral stability – Works in conjunction with the concrete cores to the north-east and south-west of building.

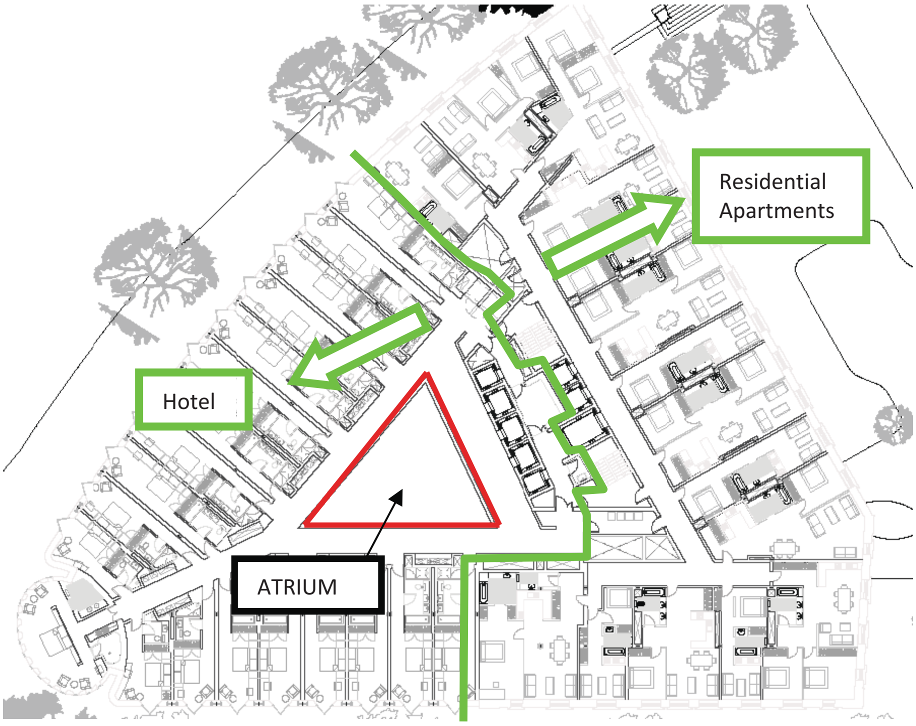



Fig. 27.1 – Typical floor plan of building

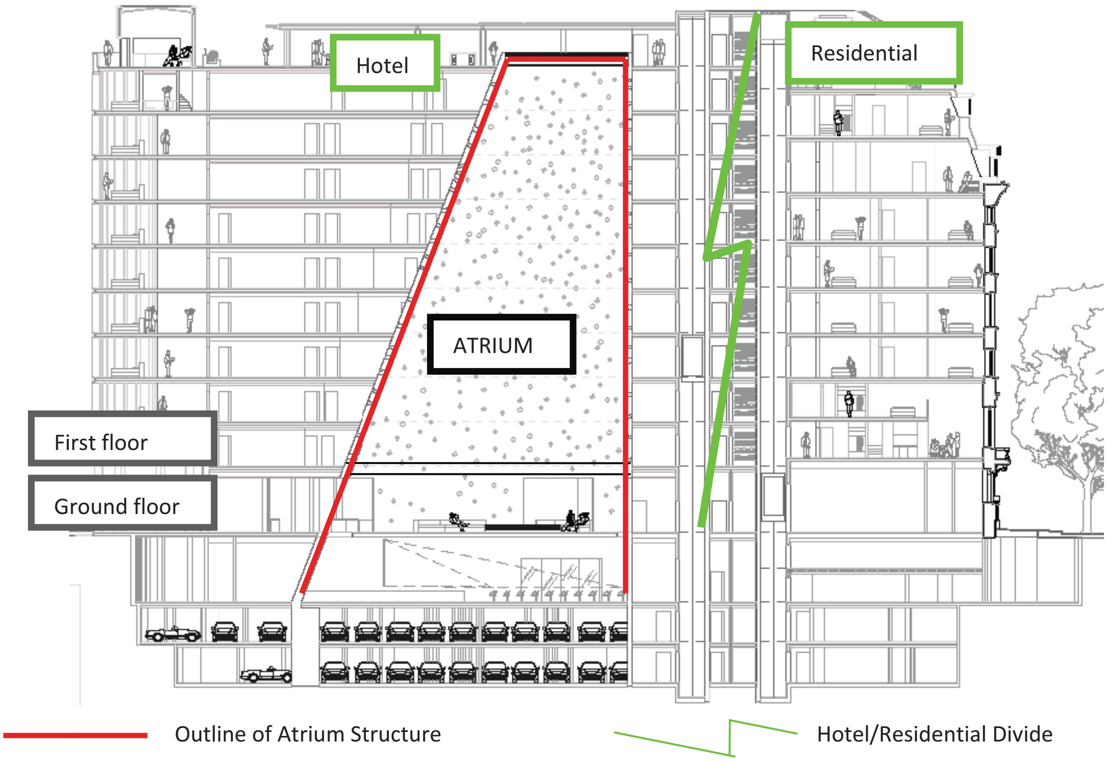



Fig. 27.2 Elevation of building

Prescriptively in accordance with Approved Document B (ADB), the atrium steelwork is required to achieve a 120-minute fire resistance period. However, within the atrium the remoteness of the steelwork from potential fire load, the protection of the stone cladding system and the provision on an automatic sprinkler protection system opens up the possibility of a performance-based design whereby only a limited amount of additional fire protection might be required for the atrium steelwork. This case study aims to address this.

## 27.2 STRUCTURAL FIRE ENGINEERING ASSESSMENT

A performance based design is adopted to determine the amount of fire protection needed for the atrium structure. Realistic fire scenarios and the risk they posed are considered based on the use of the building. The performance of the structure is then analysed considering thermal effects such as expansion and degradation of material strength and stiffness. The results are assessed against predefined acceptance criteria to ensure a safe solution.

### 27.2.1 Methodology

In principle, the study is carried out in three stages:

1. hazard identification and risk assessment
    - list all possible fire scenarios
    - undertake risk assessment and reduce risk if possible
2. thermal response modelling at elevated temperature
    - define design fire
    - calculate the heat transfer to the structure
3. structural response modelling at elevated temperature
    - calculate the response of the structure at the elevated temperature

For this case study emphasis is placed on the second and third stages.

### 27.2.2 Acceptance criteria

The acceptance criterion for the assessment is for the structure to maintain its global stability throughout the design fire duration. Local damage and failure of structural elements is therefore acceptable where this failure does not adversely affect the global stability of the structure. The rate of vertical deflection of the structure will be used as an indicator of stability. A rapid increase in vertical deflections is commonly associated with a loss of stability.

### 27.3 DESIGN FIRE

A hazard identification and risk assessment has been carried out and the worst realistic fire scenario is an unsprinklered fire at the atrium. The atrium base fire is used in the finite element analysis to determine the required fire protection scheme; as the more onerous design fire this will result in a conservative design for all other design fires. The base of the atrium space would be at first-floor level, where it is intended to be used as a reception and check-in area.

Due to the height of the atrium the fire cannot be controlled with sprinklers. A fuel load restriction is imposed on the atrium base to ensure the peak heat release rate is no more than 2.5MW for the purpose of the atrium smoke control system. Below are the details of the fuel load to be used:

- Fire loads should be placed in islands containing a maximum of 185kg of combustible material,
- The area of any one fire load island should not exceed $10m^2$, and
- The islands should be separated by a minimum distance of 3m.

The design fire was modelled as $\alpha t^2$ squared growing fire of moderate growth rate. For the purpose of the structural assessment the heat release rate has been capped at 2.7MW and no decay phase has been considered to provide a degree of conservatism. This has been based on experimental data provided in the BRE Design Fire Guide (2002). For the growth rate and heat release rate defined the total time taken to consume the available fuel within an island is 30.5 minutes. The structural elements that need to be protected are therefore provided with 60-minute fire rated protection. The performance of the structure was analysed for the full duration of the fire resisting rating. Figure x.3 shows the design fire heat release rate.

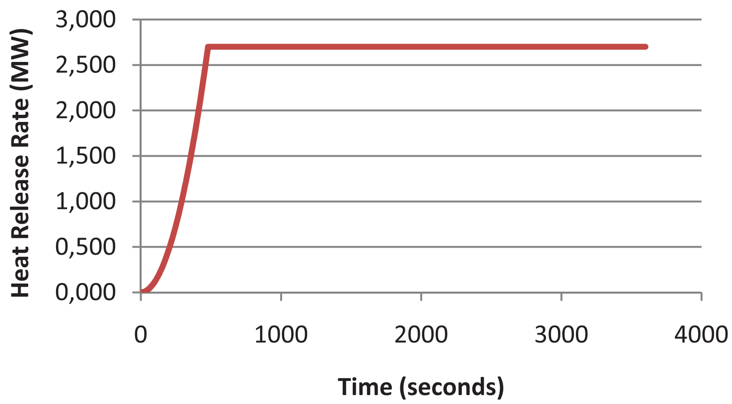


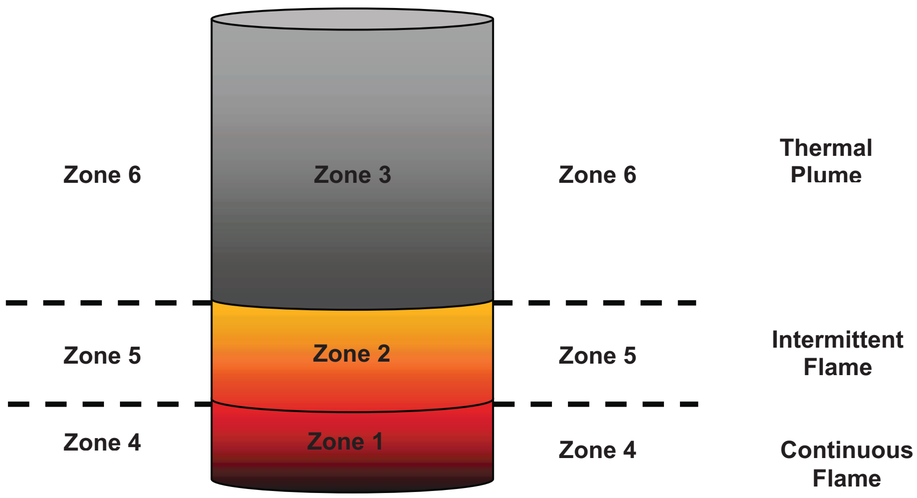

Fig. 27.3 Design fire heat release rate

### 27.4 THERMAL RESPONSE MODELLING

#### 27.4.1 Heat transfer analysis approach

The large volume of the atrium, limited fuel load and provision of smoke ventilation will prevent the smoke layer from becoming hot enough to lead to flashover. The fire therefore remains localised with high gas temperatures also localised to the fire base. The thermal exposure of structural elements remote from the fire is therefore dominated by radiation.

A localised fire heat transfer model which represents the fire as a layered cylinder of different temperatures was therefore employed. For this scenario the cylinder is composed of three temperature layers based on the geometric components of a localised fire: the continuous flame, the intermittent flame and the thermal plume. The atrium space is then divided into 6 zones based on the three geometric components of the fire as illustrated in figure 27.4




| | |
|---|---|
| **Zone 1** | Within the continuous flame – A cylinder of radius equal to that of the base of the flame |
| **Zone 2** | Within the intermittent flame – A cylinder of radius equal to that of the base of the flame |
| **Zone 3** | Thermal Plume above the flame – A cylinder of radius equal to that of the base of the flame |
| **Zone 4** | Adjacent to the Continuous Flame – At the same elevation as the continuous flame but at a radius greater than the base of the flame |
| **Zone 5** | Adjacent to the intermittent Flame - At the same elevation as the Intermittent flame but at a radius greater than the base of the flame |
| **Zone 6** | Above and Adjacent to the flame - At an elevation above the flame and at a radius greater than the base of the flame. |

Fig. 27.4 Cylinder model for heat transfer analysis

Using the cylinder model described above it is permissible to calculate the net heat flux from which protected and unprotected steel temperatures are calculated using the lumped mass approach of Eurocode 3 Part 1.2 (Section 4.2.5.1). Intumescent paint is to be used to provide fire protection. For the protected members artificially increased densities have been used to reduce the rate of temperature increase mimicking the insulating effect of the intumescent paint. This is referred to here as pseudo density.

The appropriate pseudo density was calculated using a goal seeking approach, where the goal is set as the assumed steel section failure temperature (620ºC) at the prescribed fire resistance rating of the applied intumescent paint (FR60). It has further been assumed that after reaching the fire resistance period the fire protection will fall off completely and therefore the element will be treated as unprotected thereafter. Figure 27.5 contrasts the standard temperature time curve and temperature evolution for an unprotected and intumescent 60 min protected element.

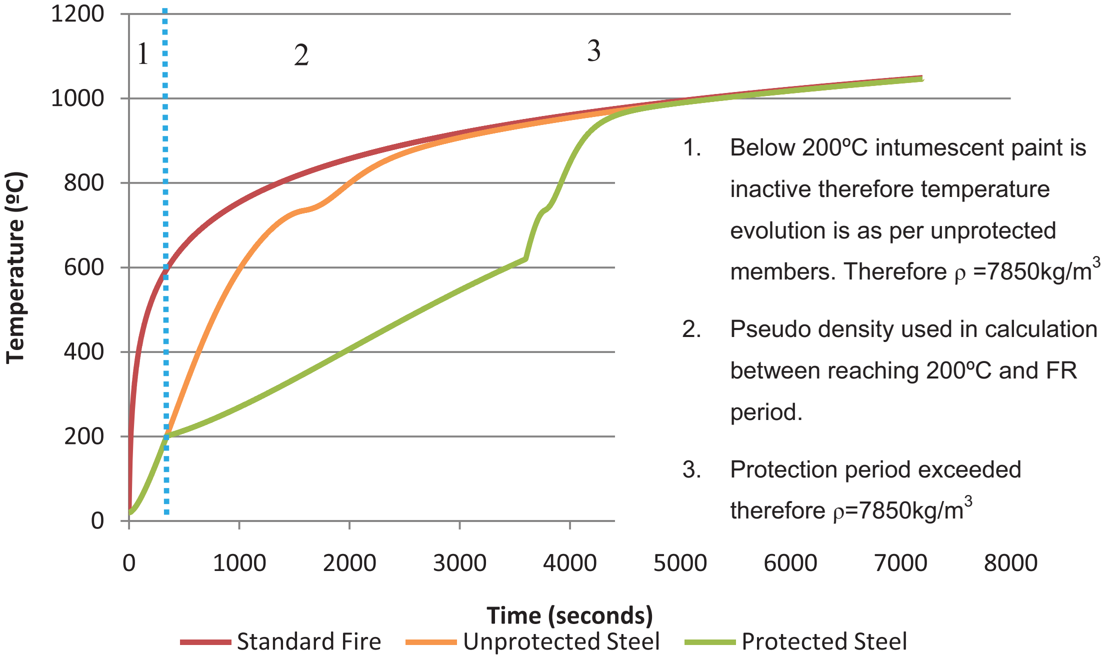



Fig. 27.5 Protected and Unprotected steel temperature evolution

### 27.4.2 Assessed fire scenarios

It has been identified that the vertical corner elements (legs) of the frame are critical elements for frame stability. The worst case location of the fire on the atrium base is therefore in close proximity to a leg. Symmetry allows us to consider just two fire locations. The performance of the atrium frame is analysed under four different fire and loading combinations. These are summarised along with the indicated performance result in Table 27.1.

Tab. 27.1 Analysis Combinations

| Fire\ Loading | Dead + Live | Dead + Live + Wind |
|---|---|---|
| Nose | Maintains Global Stability | Maintains Global Stability |
| Rear North | Maintains Global Stability | Maintains Global Stability |

### 27.4.3 Determination of fire protection scheme

The determination of the required protection scheme was conducted in an iterative fashion. From interrogation of the frame to identify key elements for stability the following fire protection scheme is proposed as a first estimate:

- atrium steelwork between first floor level and second floor level the protection is applied as outlined in Fig. 27.6 – these elements are likely to receive large amounts of radiation in the event of a fire

- column at apexes from second floor onwards - 60 minutes fire protection – these elements have been identified as critical for the structural stability of the frame

If the analysis results do not achieve the acceptance criterion (mentioned in Section 27.3.2) the frame is assessed for points of weakness by the user. The analysis is then re-run providing protection to this area.

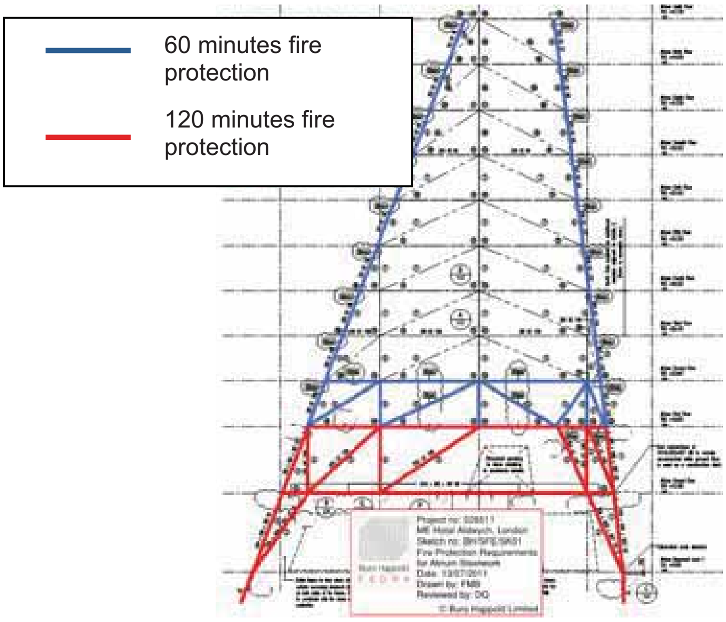



Fig. 27.6 – Fire protection layout on atrium structure

#### 27.4.4 Heat transfer to structural members

The analysis of the thermal response of the structural frame does not include the beneficial shielding or insulating effects by the proposed marble cladding system mentioned in Section 27.2.3 as it was not possible to guarantee performance of the stone cladding in the areas where the direct flame impingement could be possible.

The thermal response of each individual structural element has been calculated using the methodology described in Section 27.6 for the nose and rear north corner design fires. This calculation considered the effect of the intumescent paint 60 minute fire protection applied to the steelwork stated in Section 27.4.3. Figure 27.7 illustrates the atrium frame temperature profile after 60 minutes exposure for a fire located in the (a) nose of the atrium and the (b) rear north corner. The majority of the atrium frame remains quite cool (< 100ºC) for the duration of exposure. The figure caps the temperature contours at 220ºC to illustrate the extent of area where the intumescent paint if present is activated; these are depicted in grey.

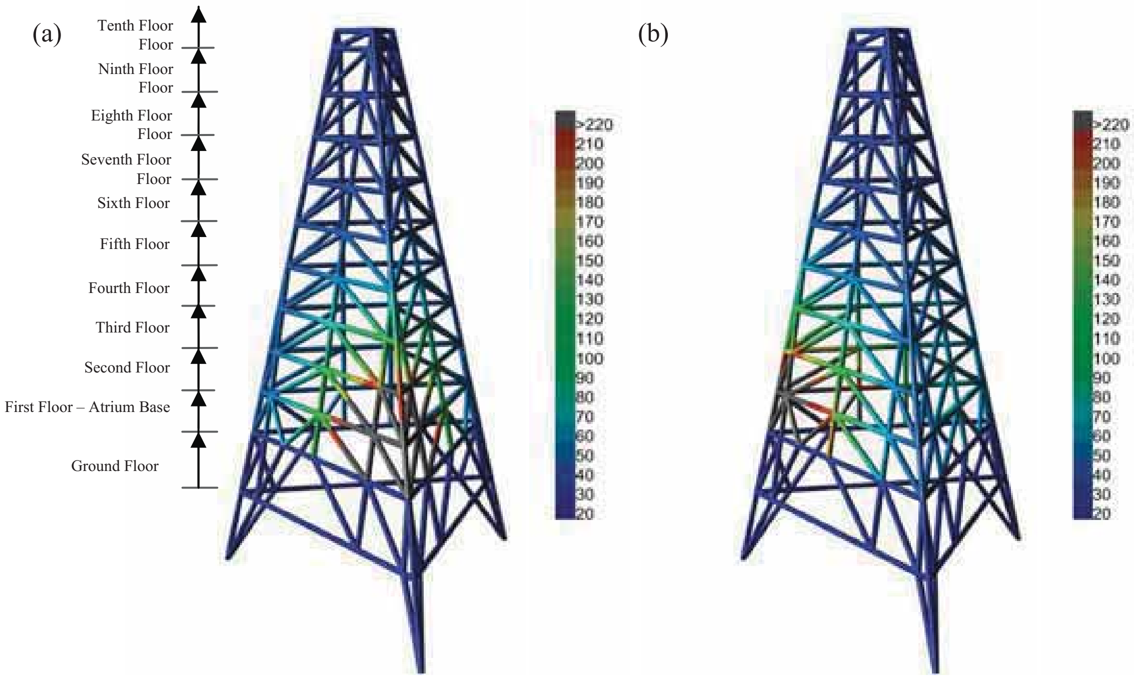



Fig. 27.7 Structural Frame temperature profile 220ºC cap (ºC) after 60minutes (a) Nose Fire (b) Rear North Fire

## 27.5 STRUCTURAL RESPONSE MODELLING

### 27.5.1 Structural analysis methodology

The structural analysis was conducted using the finite element (FE) software VULCAN, a special purpose high-temperature structural analysis FE package. The acceptance criterion for this assessment is for the structure to maintain overall stability for the duration of the previously defined design fire. Local damage and failure of structural elements is therefore acceptable where this failure does not adversely affect the global stability of the structure. The rate of vertical deflection of the structural elements within the frame will be used as an indicator of stability; a rapid increase in vertical deflections is commonly associated with a loss of stability.

### 27.5.2 Description of Vulcan model

Vulcan was used to carry out the finite element analysis, using mainly beam elements. The geometry of the structure was been obtained from the structural engineers and imported into Vulcan.

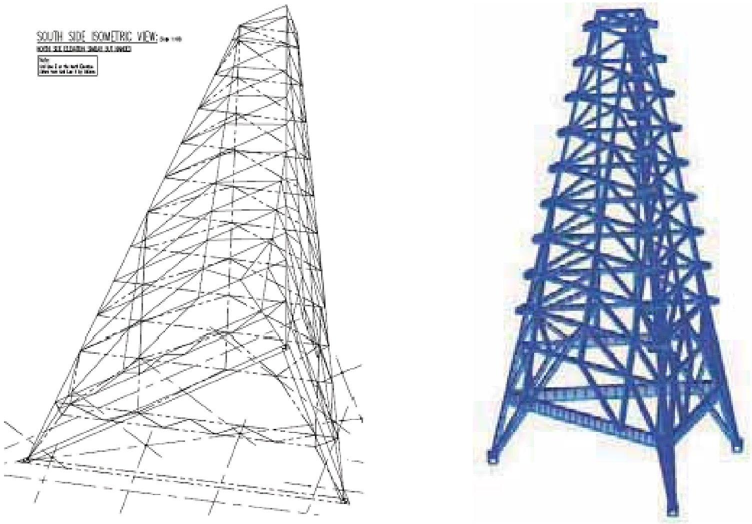



Fig. 27.8 Structural Frame (a) Isometric view (b) Vulcan view

### 27.5.3 Loading

For the structural analysis, load factors at fire limit states used for the design are as stipulated in BS5950 Part 8, which are 0.8 for imposed loads, 1.0 for dead loads and 0.33 for wind loads.

### 27.5.4 Modelling effects of restraints

The vertical component of the base support is modelled as fixed, that is infinitely stiff. The lateral components are restrained using directional springs.

The atrium frame is restrained by the surrounding structure at each level; thermal expansion is known to have both beneficial and detrimental effects it is therefore necessary to include the effects of this restraint within the VULCAN model.

Firstly, the floor slabs of the surrounding structure are tied to the horizontal frame members as indicated in Figure 27.9(a). The restraining effect of the floor slabs for the horizontal members has been represented by the inclusion of small width (1m) slab elements which are tied to the perimeter beams. These slab elements are non-load bearing, their effect upon the model was limited to the provision of longitudinal restraint of the perimeter beams. The location of these slab elements is illustrated in Figure 27.9(b), where the slab elements are depicted in red. Slab elements were only applied from second floor onwards, as the steelwork from first floor downwards are not going to be affected by the fire at the base of the atrium.

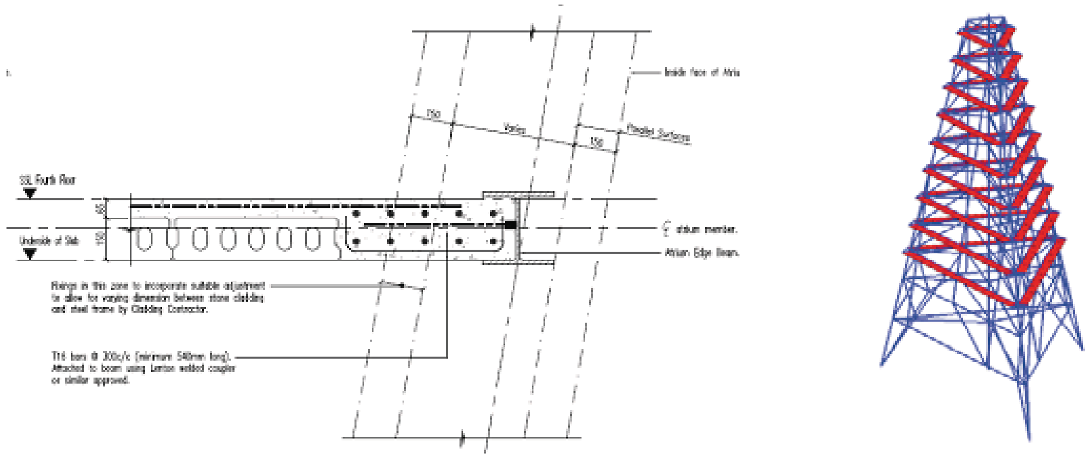


Fig. 27.9 (a) Edge beam to floor detail (b) Slab panels providing lateral restraint

Secondly, the atrium frame is a component of the buildings lateral stability system. The building has been designed to very low lateral movement tolerances (height/1000) to minimise any damage to the retained historic façade. The building's lateral stability system comprises three elements, a curved concrete wall on the western façade, the atrium steel frame and a central concrete core constituted by two shear walls. The location of these on a typical floor layout is illustrated in Fig. 27.10(a).

To gain a realistic assessment of the performance of the structure in the event of a fire to include the lateral restraint imposed by the concrete west wall and central core upon the atrium. This restraint to the atrium frame has been modelled as a stiff column with a fixed base tied to the atrium floor at each floor level. In Figure 27.10(b) the location of the column and the sections that tie it to the atrium frame are depicted in red. The bending stiffness at each level has been deduced from the atrium frame structural calculations.

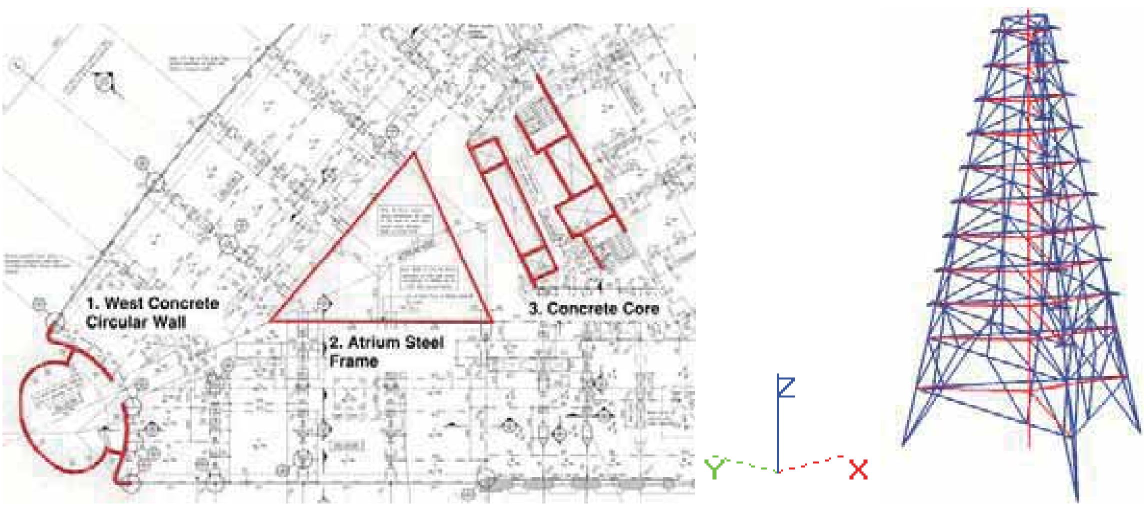



Fig. 27.10 (a) Location of core walls (b) VULCAN Model Core Representation

### 27.5.5 Structural response

The vertical deflections at the three corners of the 10th floor level were used to check the rate of deflection during the fire exposure; these locations are illustrated in Fig. 27.11.

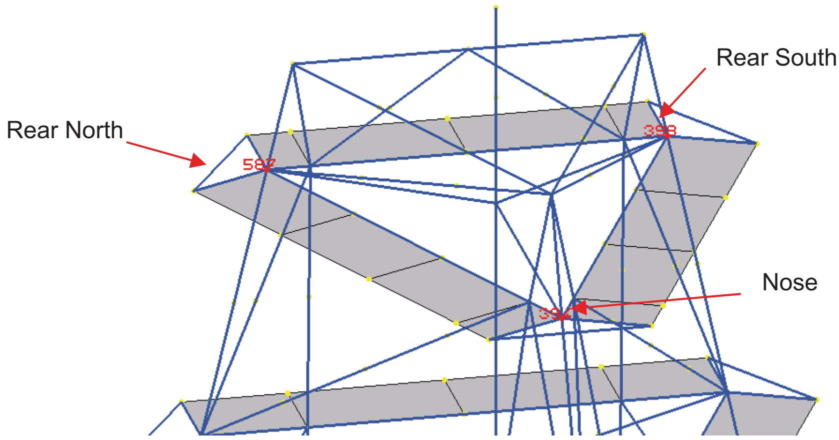



Fig. 27.11 10th floor locations of vertical deflection measurement

<u>Rear North Corner Fire</u>

Figure 27.12 graph the evolution of the 10th floor vertical displacement for the duration of exposure for the rear north corner fire under dead, live & wind loading respectively. The vertical deflection of the atrium frame under a fire located in the rear north corner is characterised by a relatively quick increase in deflections from about 5 to 15 minutes for both load combinations. Quickly increasing deflections in the early stages of exposure are related to the rapid heating of structural elements local to the fire to 200ºC whereupon the intumescent fire protection activates. During this time some buckling of isolated members at first floor level is evident; however as for the nose located fire the subsequent decrease in deflection rate indicates that this buckling does not adversely impact upon global stability.

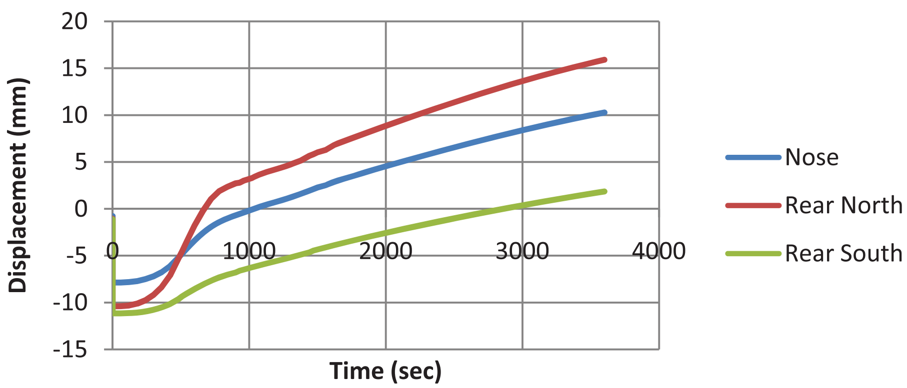



Fig. 27.12 Rear North corner under dead, live & wind loading – 10th floor vertical displacements

Deformation plot

Figure 27.13 shows the deformation and local buckling occurring to the structure of the structure at 60 minutes for a dead and live load combination.

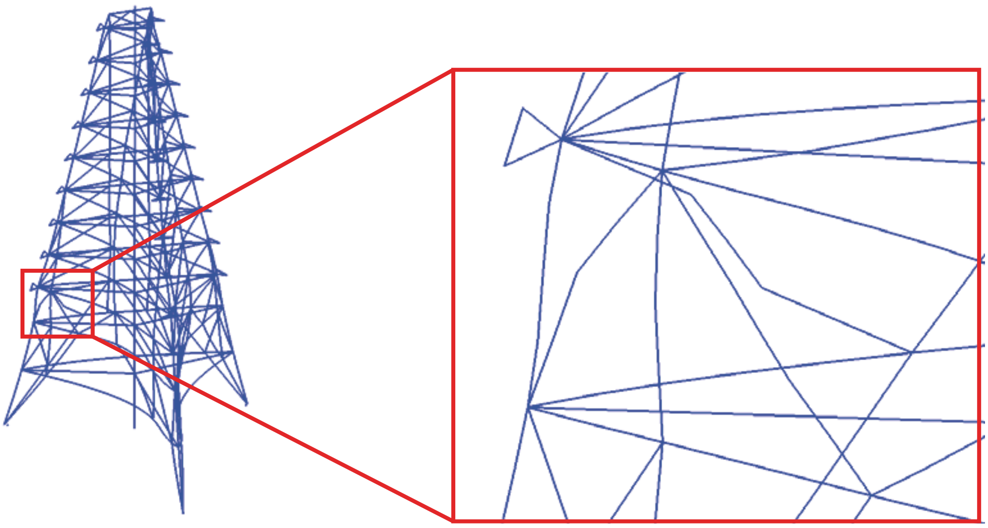


Fig. 27.13 Deformation plot for nose fire scaled at 35.

### 27.5.6 Discussion of results

The results of these analyses indicate that the structure maintains global stability during exposure to a localised fire subjected to the thermal analysis and structural analysis described under Section x.4 and Section x.5 respectively. This implies that the preliminary fire protection scheme stated in Section x.4.3 is sufficient in ensuring the load carrying capability of the atrium structure in an event of a fire, despite some buckling of steel members at the vicinity of the fire.

## 27.6 POST-STUDY

A report based on the findings in this study was submitted to the local building authority and has been subsequently approved. The fire protection scheme, as indicated in Fig. 27.6 was communicated to the steel fabricator, whereby the amount of fire protection needed to achieve the required fire rating is calculated in accordance to BS476.

## 27.7 SITE IMAGES

As this case study was written, construction on the building was in its final stages. The proposed fire protection regime outlined above was followed, with the exception of the steelwork between first floor level and second floor level, in which all members were fire protected. Below are some pictures of the atrium steelwork.

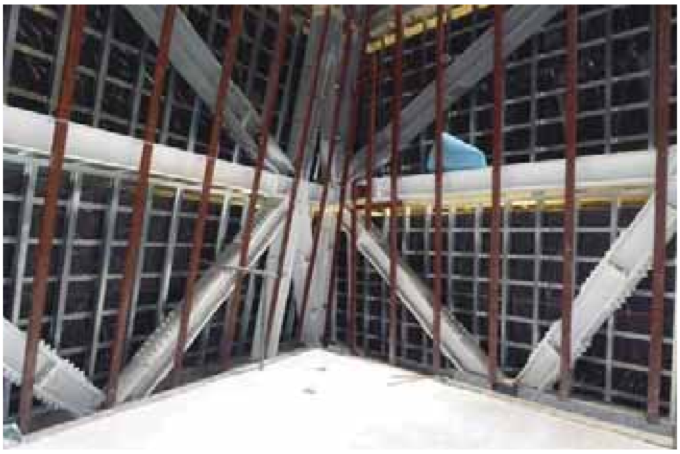


Fig. 27.14 Intumescent paint sprayed on site at first floor

**27.8 CONCLUSIONS**

The paper summarises a structural fire assessment carried out on an atrium structure. While the prescriptive guide recommends the whole structure to be protected by a 120-minute fire ratings, engineering judgement suggests that the structure could be partially protected without compromising the stability of the structure in an event of a fire. A hazard identification and risk assessment was carried out on the possible fire scenarios. After identifying an unsprinklered fire at the atrium base as the worst case and assuming a preliminary fire protection scheme, a heat transfer analysis was then conducted. The results of the thermal analysis implies that compartmentation on two floors could be breached therefore a 60-minute compartment walls are proposed on those floors. Considering thermal effects, the structure was then analysed in Vulcan subject to different fire positions and different load combinations. The predicted structural response indicates that the global stability of the structure is maintained throughout the duration of the fire period. Thus it has been demonstrated that only certain members require fire protection (60 minutes within the atrium or 120 minutes out with the atrium) while majority of the steelwork within the atrium can be left unprotected. This case study demonstrates that combined rationalisation of the potential fire scenarios and consideration of global structural performance can produce an optimised structural fire protection scheme which meets the life safety objectives of the Building Regulations.

Acknowledgement

The author would like to thank Meliá Hotels International and Fosters & Partners for their permission to present this work. Meliá Hotels International is the hotel developer based in Palma de Mallorca, Spain.
Information about the company can be found via MELIA.COM, facebook.com/MeliaHotelsInternational or twitter.com/MeliaHotelsInt.

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Tom Molkens, STUBECO, Belgium

# 28 FIRE ENGINEERING APPROACH – CASE STUDY "DE POSTHOORN" AT HAMONT

Summary

In 2007, a new transformation and extension of the existing hall was done. Due to a misunderstanding about the regulations, there was no fire protection at all foreseen for the transformation. The new part of the building that contains a stage needed to fulfil the latest legal requirements. In this case, this implies a safety level equal to that associated to a fire resistance of 30 minutes ISO834.This became clear after the stage was fully equipped with all technical installations.

## 28.1 INTRODUCTION

The building of this case study is an existing candle factory which had already been transformed in the past to a local village hall. Because this previous transformation was performed before implementation of regulations concerning safety and fire resistance, there was at that time no special measure taken about this issue.

In 2007, a new transformation and extension of the existing hall was done. Due to a misunderstanding about the regulations, there was no fire protection at all foreseen for the transformation. The new part of the building that contains a stage needed to fulfil the latest legal requirements (Fig. 28.1). In this case, this implies a safety level equal to that associated to a fire resistance of 30 minutes ISO834.This became clear after the stage was fully equipped with all technical installations.

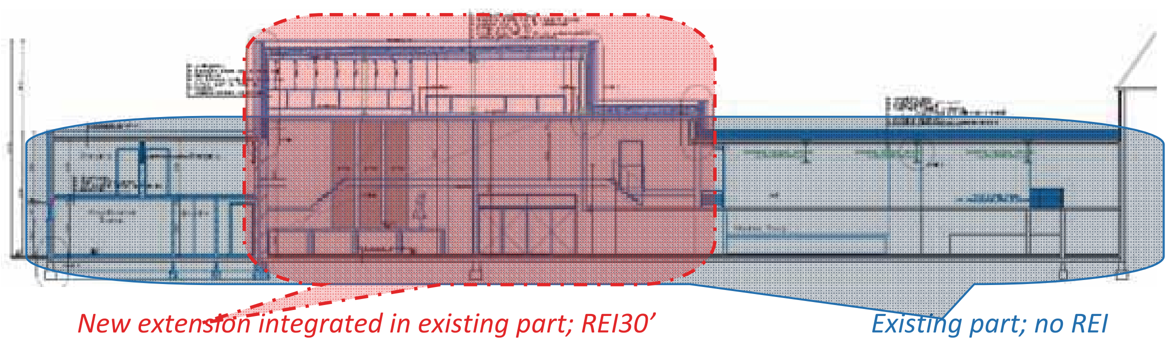


Fig. 28.1 Section over the whole building

Because the renovated hall is located in the centre of the village, also adequate acoustic insulation criteria needed to be fulfilled. Those are the most stringent for the stage and for that reason a massif 30 cm concrete roof was needed. To support this roof two main beams where needed, spaced at about 4,85 m with a span of 22,38.

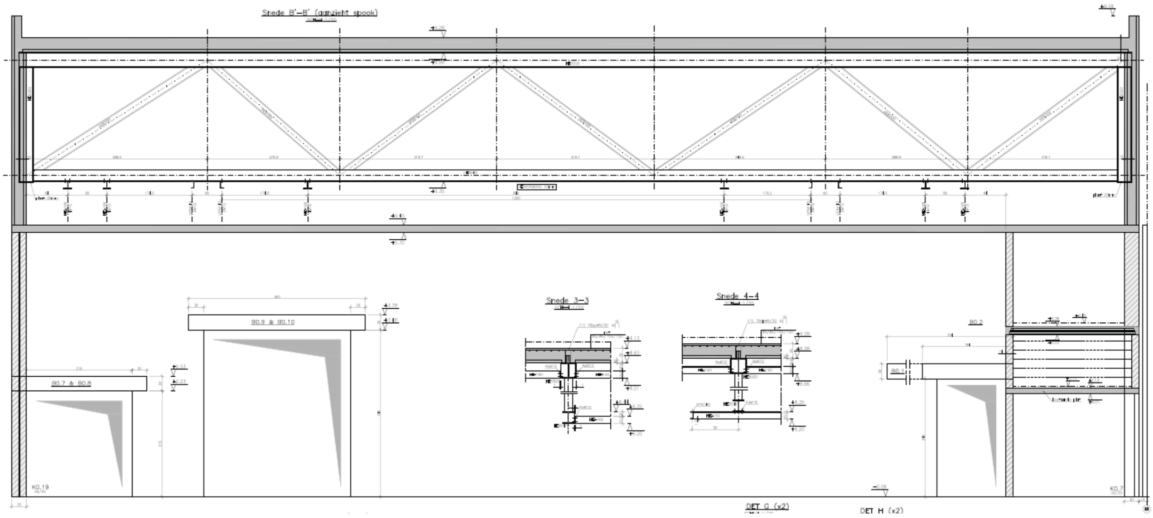

Fig. 28.2 Section of the stage

To avoid a building height higher than 10 m and to make the technical equipment accessible by the aid of catwalks, a steel truss girder with irregular diagonal slopes was chosen. A concrete beam would indeed be rather heavy and couldn't make an access possible with the catwalks because it would be too deep (about 2 m).

## 28.2 ORIGINAL DESIGN

For the design of the steel truss girders, the framework software "Power frame" from the company Buildsoft was used. Deadweight and variable loads from the roof act on the upper chord of the truss girder; loads are also applied on the secondary beams supporting the catwalk.

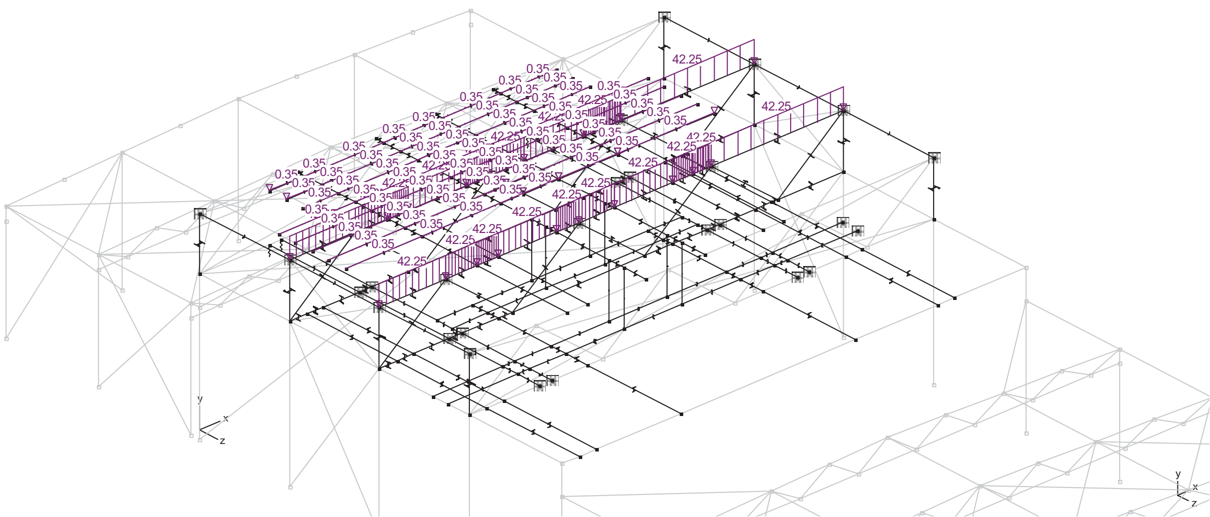

Fig. 28.3 Dead loads acting on the steel structure

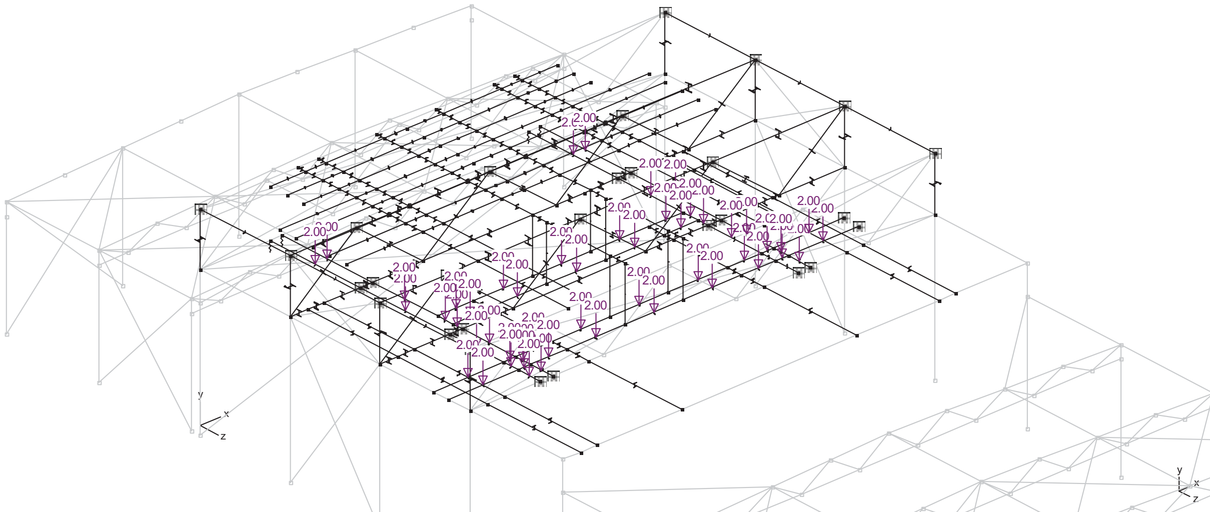

Fig. 28.4 Mobile loads acting on the steel structure, considered as concentrated loads

Effects of wind and of the own weight of the structure were also taken into account. Verification was done for service and ultimate limit states. A summary of the applied loads can be found in Tab. 28.1. Verifications resulted in a resistance ratio of 96.2% and a stability ratio of 98.9%, both number being given for the most critical element in the structure, see Fig. 28.5.

Tab. 28.1 Loads acting on the steel structure

| Load | Ψ | kN/m² | Continuity factor | Service load in kN/m | Load in case of fire kN/m |
|---|---|---|---|---|---|
| Own weight | 1 | Variable | - | Variable | Variable |
| Dead load | 1 | 0,3×25+0,28 | 1,12 | 42,25 + tracks | 42,25 + tracks |
| Snow load | 0 | 0,5×0,8=0,4 | 1,12 | 2,18 | 0 |
| Mobile load, maintenance | 0 | 1-A/100 or concentrated 2kN | - | Conc. Loads 2 kN | 0 |
| Wind actions | 0.2 | 0,63×(0,75+0,3) | 1,0 | 3,21 | 0 |

For the mobile loads, it appeared to be more critical for the catwalks to consider the concentrated load instead of the distributed loads, because of their small surface. For the main beams on the other hand, where the surface is higher than 60 m², the surface load was considered. The dead load is composed by the own weight of the concrete roof slab spanning on four supports+ thermal insulation + watertight membrane. This load is considered with a continuity factor in order to take into account the effect of the three unequal spans in the concrete roof.

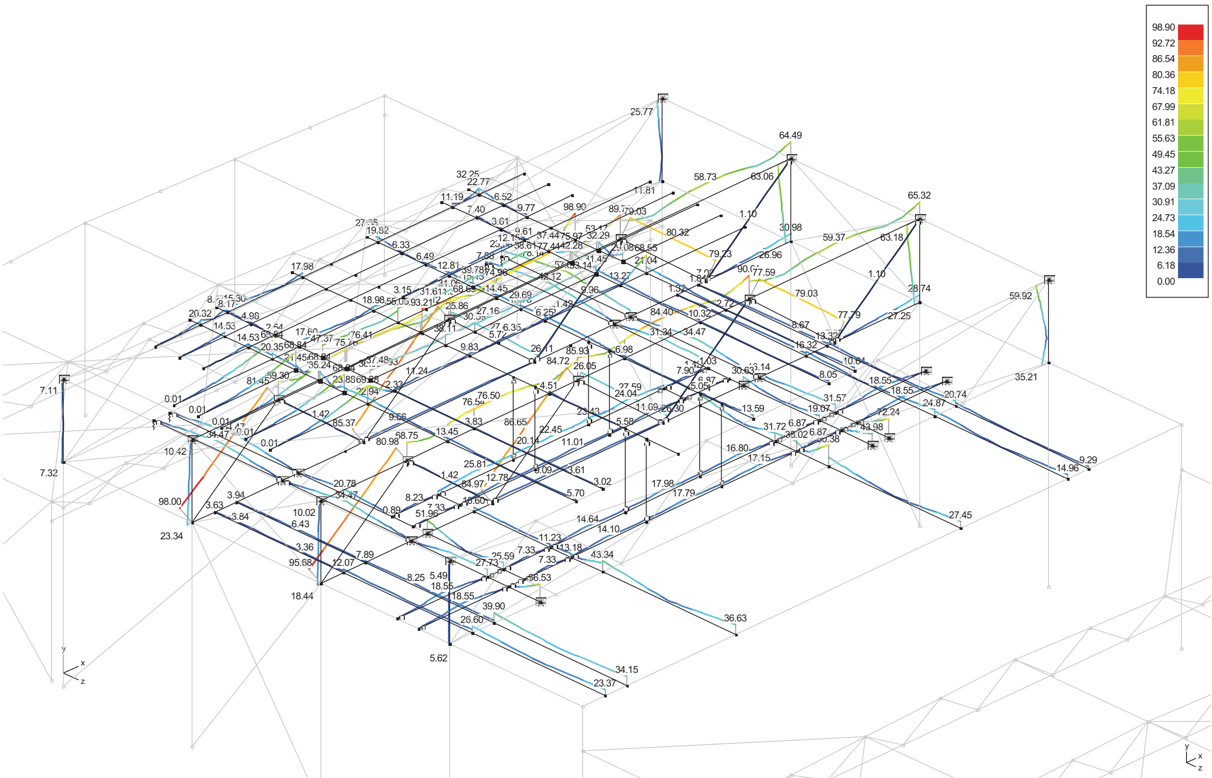


Fig. 28.5 Result of the stability check

## 28.3 FEASIBILITY STUDY

To get a first idea of the stability in case of fire, the thermo elastic module of the same software was used but it became immediately clear that it is not possible to prove a stability of 30 minutes with a thermo elastic model because the restraint to thermal expansion generates huge stresses. In this building especially, there is significant restraint caused by the concrete elements.

After this first attempt, a more advanced finite element model was used for the analysis of the structure. Concrete walls are modelled with plate and wall elements but without membrane effects. Catwalks are not considered in the model as a steel members but as an extra fixed load of 1,75 kN/m on the upper chord of one truss girder. Only the fact to consider the collaboration between steel and concrete (in bending) we could already improve the resistance and stability verification checks. The values presented in Fig. 28.5 are reduced to36 % and 56 % for the most critical elements and same loads.

The hypothesis is made that concrete will be hardly affected by a 30 minutes fire. Another appreciation of the behaviour of the structure was obtained by affecting the strength and stiffness of the S235 steel in the model mentioned above by the reduction factors mentioned in Table 3.1 of EN 1993-1-2.

This model shows that the secondary elements are working in compression, see Fig. 28.8.

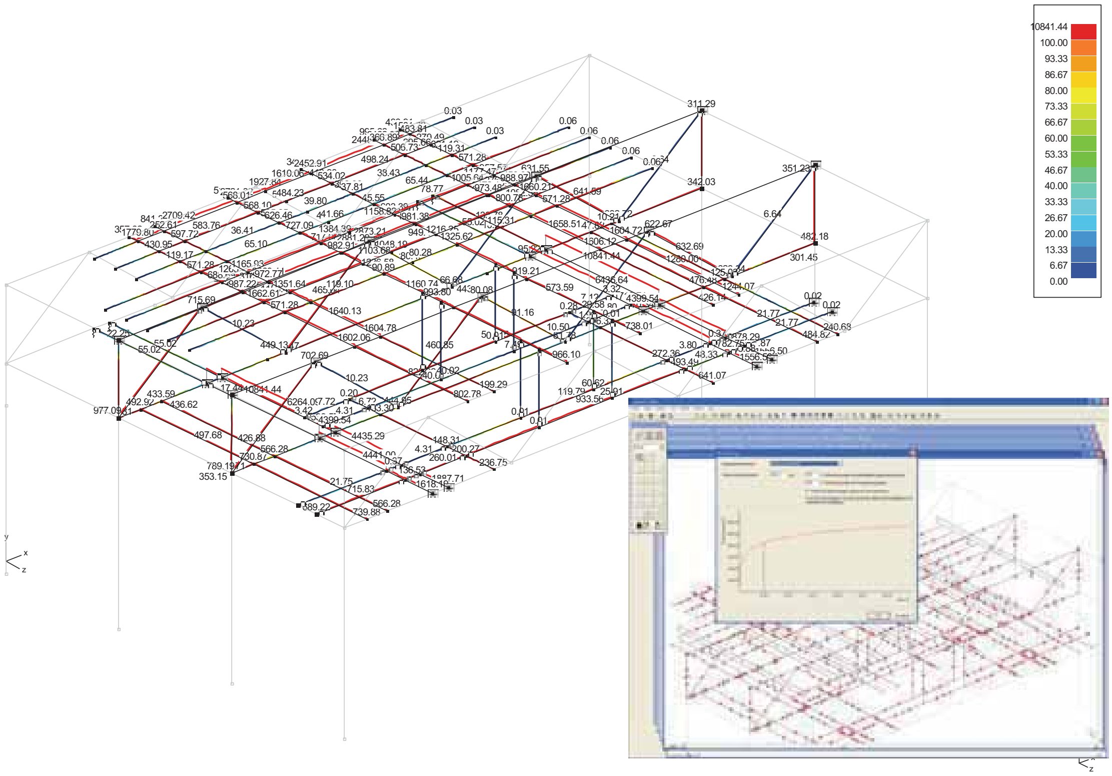


Fig. 28.6 Result of the resistance/stability check with a thermo elastic approach

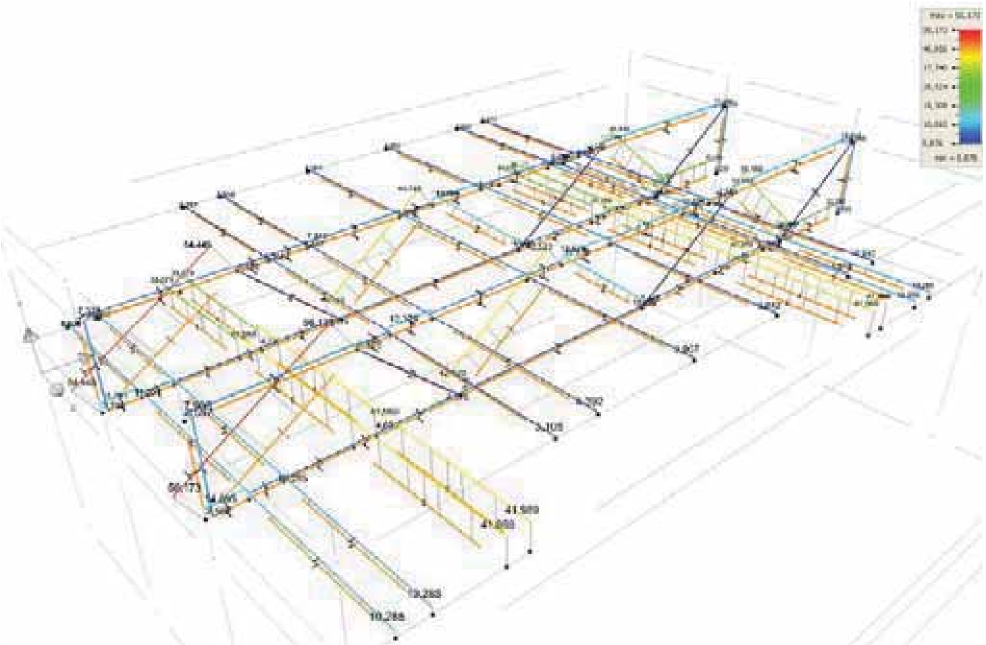

Fig. 28.7 Result of the resistance/stability check

Tab. 28.2 Values from table 3.1 EN 1993-1-2

| | $k_{y,\theta} = f_{y,\theta}/f_y$ | $k_{p,\theta} = f_{p,\theta}/f_y$ | $k_{E,\theta} = E_{a,\theta}/E_a$ |
|---|---|---|---|
| 700°C | 0,230 | 0,075 | 0,130 |
| 800°C | 0,110 | 0,050 | 0,090 |
| 900°C | 0,060 | 0,0375 | 0,0675 |

Tab. 28.3 Temperature in steel after 30 minutes of ISO834 fire

| Profile | exposed sides | Temperature °C | Effective yield strength N/mm² | Youngs modulus N/mm² |
|---|---|---|---|---|
| UPN140 | 4 | 825 | 22,9 | 17719 |
| HEA160 | 4 | 824 | 22,9 | 17719 |
| HEB160 | 4 | 801 | 25,9 | 18900 |
| HEB200 | 4 | 784 | 25,9 | 18900 |
| HEB240 | 4 | 768 | 34,9 | 21588 |
| HEB320 | 3 | 746 | 41,1 | 23436 |

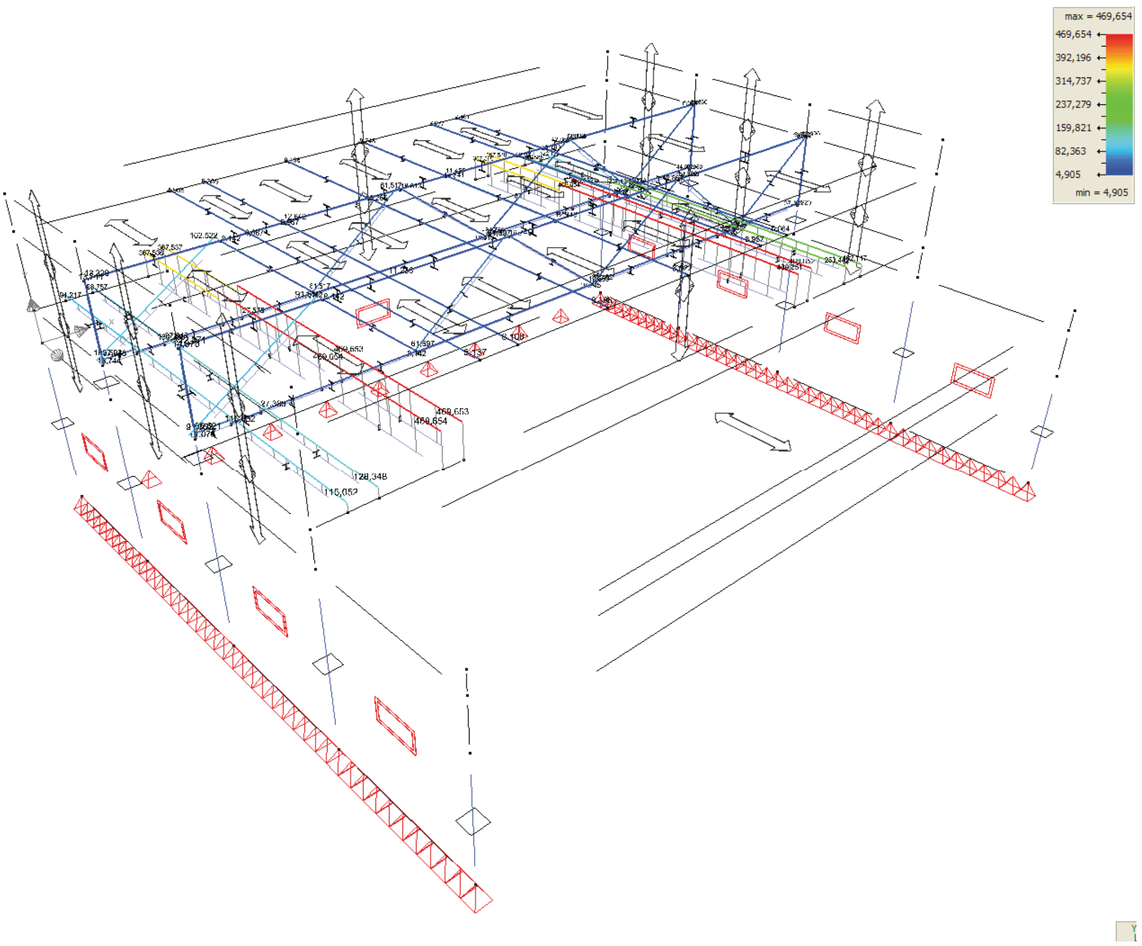


Fig. 28.8 Result of the resistance/stability check with reduced stiffness

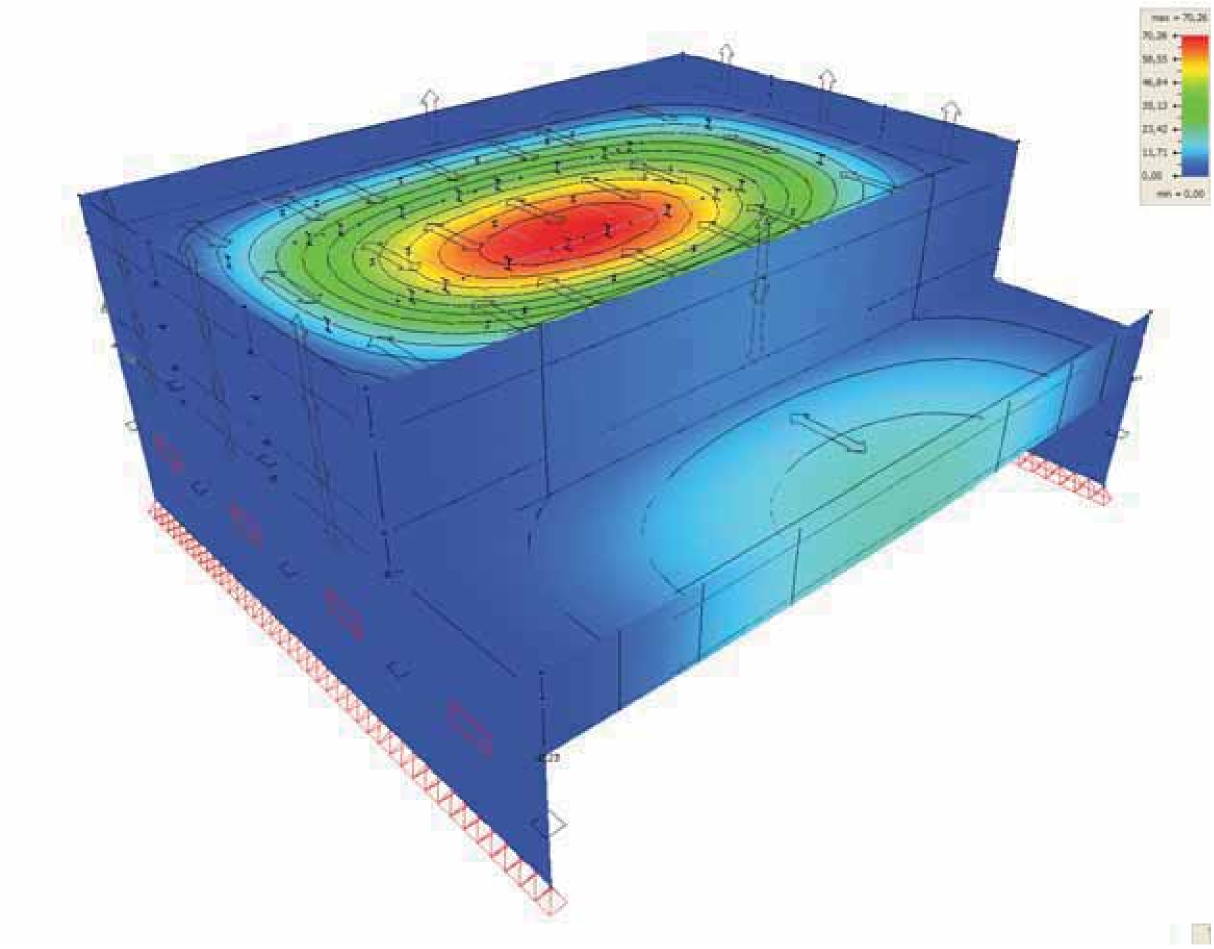

Fig. 28.9 Deflections with reduced steel stiffness

It could be expected that rather high deformations will occur. This is in fact the case as can be seen on Fig. 28.9 where the roof deforms in a membrane like shape. This model showed that the reinforcement calculated from the bending moments on an elastic way is much higher than what had been foreseen: Lower 450/141 mm²/m and upper mesh 150x150x8x8.

## 28.4 NATURAL FIRE DESIGN

The temperature development in the compartment was estimated with the two zone model Ozone, Cadorin and Franssen (2003), Cadorin et al. (2003). A fire curve obtained by this model is shown on Fig. 28.10, together with the temperature development in a HEB240 steel section subjected to this fire curve.

## 28.5 FINAL STUDY

The simple analysis model is not able to prove stability of the structural system, even in the case of a natural fire solicitation. With the large deflections that are observed with the simple model, it is clear that tensile membrane action in the concrete slab will take place and should be considered in the load bearing capacity, which is not possible in the simple model (leading to excessive amount of reinforcement). It will thus be necessary to use a more refined structural analysis tool that would be able to take into account the effects of tensile membrane action.This is the case whatever the fire scenario that is being considered, either the ISO834 fire curve or a natural fire curve.

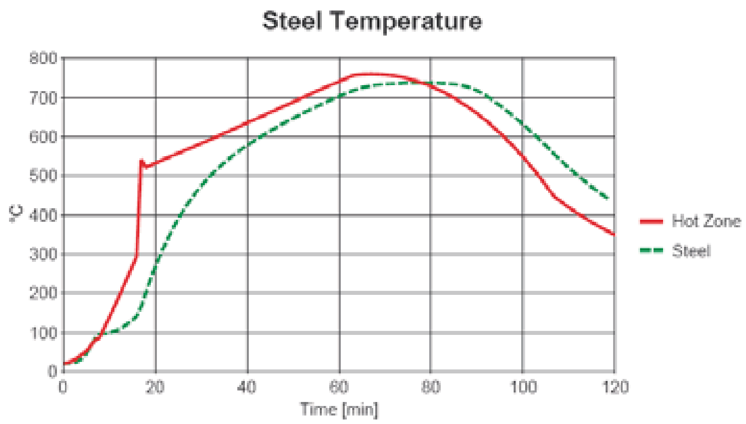


Fig. 28.10 Result of Ozone model

**28.6 CONCLUSIONS**

The analyses were conducted on a structure that is essentially made of a concrete slab supported by two unprotected steel girders. An analysis based on thermo-elastic model confirmed that such model is totally inadequate when some degree of restraint is involved. A second model was built in which no degradation was considered in concrete and the strength of stiffness of steel where modified according to the reduction factors of Eurocode 3. This model working in small displacements is not able to tackle the tensile membrane action that develops in the slab due to the large displacements that occur when the steel truss girders lose their strength and stiffness.

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# 29 RECONSTRUCTION OF WAREHOUSES IN BUDAPEST IS THE BIRTH OF CET

Summary

In this paper the case study about the construction of the CET building in Budapest is presented. CET is a mixture of historical and new architecture, it is a 160 meter long "whale" of steel, glass, and aluminium, which is situated among Budapest's historical buildings. The historical part is the rest of the Warehouses, completed in 1881 and remained one of the capital's main distribution centres until World War II. The Warehouses was damaged by heavy bombardments and the area didn’t develop anymore. The reconstruction was an idea to save this historical monument. The basic of the fire concept was the performance based fire engineering, because the modern notion of the structure and building shape didn't match with the Hungarian Fire code. The two different main part of the building, a huge atrium like a retail area (the body of the whale) and a multi floors event centre (the head of the whale). Both of the smoke and heat exhaust system of the main part was analysed by the FDS. It was centralized to the circumstances of the evacuation and the temperature of the area around the structures. The result of the new view point brought more new solutions, which make reality from this building in Hungary.

## 29.1 INTRODUCTION

After the Revolutions of 1848-49 and especially after 1867 the economy of Hungary began to recover and this new period also resulted in a commercial blooming that took place in Budapest. In 1875 a decision has been made. The warehouses had to be positioned under the toll-house. Finally, after an immense amount of disputes, in 1879 the decision was born, namely the building of four storage rooms. The implementation of the plans finished on the 30th of September in 1881. These four buildings have been connected with an elevator that pumped through the corn from the steamships. The whole structure (including the neighbouring warehouses) has been destroyed during the Second World War.

The area is in its present state since 1966; one of the four storage houses has been demolished because it was greatly damaged during the bombings of the Second World War. The elevator and the neighbouring warehouse have also been demolished and Park Nehru opened there in this year.

During the last years it has been brought up that the area should be put to use once again; rebuilding the old, monumental buildings with the help of new architectural elements.

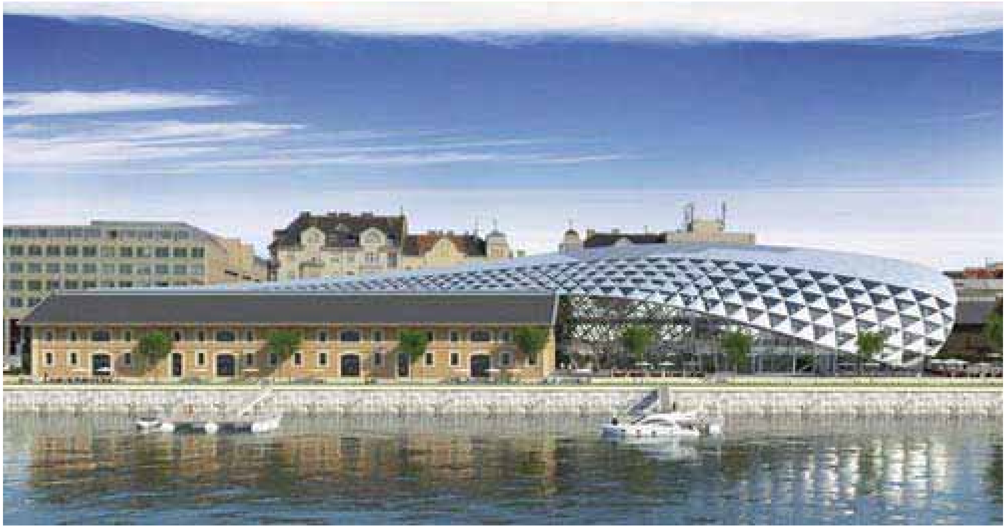

Fig. 29.1 Visualization of CET Budapest

Fig. 29.2 View of CET Budapest (Januar 2012)

Finally, the outline of the new building was born and the dilapidated warehouses have been rebuilt; CET stands there now, a new construct that could become a new symbol of Budapest. There used to be four warehouses at the bank of the River Duna in the district Ferencváros but one of them was totally destroyed during the bombings of the Second Word War.

## 2 GENERAL BUILDING DESCRIPTION

At the CET Budapest the architect designer is Kas Oosterhuis, the general main contractor WHB Építő Kft.

The name CET refers to the local time zone (Central European Time) and to the whale shape of the building (cet means whale in Hungarian). The first plans were slightly modified since the architects had to round off the whale's nose and the system of the coating was also modified but these are only minor changes. The space between the two buildings will be covered with a glass cap and this way it will look like the above mentioned animal or something like the waves of a river.

The total territory of the new building is 31000 square metres from which 12500 square metres is the usable area. What you can find here is a room with space for 1200 people that is maintained for programmes, an underground garage with 250 parking spots, galleries, book shops, restaurants, cafes and nearly 200 meter long terraces that are situated at the bank of the River Duna.

Regulations prescribe the 60% retention of the three original storage buildings. A 20 metre part of the buildings' Northern side is going to be demolished so that the future construct will be easier to approach (thanks to a 20x50 metre square). The third (Southern) warehouse will be almost totally removed.

The appropriate linking of the modern architectural forms to the older parts is a great challenge. The entrance that is made up of a glass structure conjugates the two solid brick walls. The arched glass rooftop is the main visual element, it gives an iconic appearance to the core body of the building and it also creates an inner space with a unique atmosphere that is sunlit and protected from the weather conditions. All the steel, aluminium and glass elements of the roof are different and economic to produce (thanks to computer studies). These materials have to be connected with the elements of the old building (namely wood, brick and stone).

The body of the whale will be 160 metres long and the width of the glass rooftop will be 18 metres. The glass rooftop that has a triangle cross-section with its upper part slightly leaning out touches the top of the warehouses (from the North). Heading south it slowly turns into a curve, then overhanging the warehouses it turns into a glass shade that is the whale's head.

## 29.3 FDS SIMULATION

### 29.3.1 The Hungarian Law prescribes the following:

On the basis of the Hungarian Fire Code (OTSZ) the heat and smoke exhaust system has to be as big as 3% of the atrium's floor space. Hence, a surface of this size is required for both exhaustion and air inflow. If we would like to set up the exhaust system with the help of mechanical devices then every square meter would receive 2m$^3$/s air inflow. There are two atriums within the building. One in the 'body' and one in the 'head' of CET. The surface of the 'body' structure's atrium is 1000 square meters, while the 'head' structure has a floor space of 590 square meters.

That is why a 218700 m$^3$/h suction and a 30, 34 m$^2$ air inflow space has to be provided in the 'body' and a 126500 m$^3$/h suction and a 17, 56 m$^2$ air inflow space is required in the 'head'.

According to Hungarian Fire Code (OTSZ), smoke and heat exhaustion (in the case of mechanical exhaustion) has to be started on the level of the fire and on the levels above the fire level.

Tab. 29.1

| According to OTSZ | Suction | surface for air inflow |
|---|---|---|
| 'Body' | 218 700 $m^3/h$ | 30,34 $m^2$ |
| 'Head' | 126 500 $m^3/h$ | 17,56 $m^2$ |

**29.3.2 Why was it necessary to approach the project from an engineer's perspective?**

It was problematic in both the 'body' and the 'head' structure that the required amount of air inflow could not be provided (due to the architectural conception). Only half of the ordered surface was available. The area of the 'head's atrium is interconnected by joists. We were afraid that smoke and heat exhaustion (that is started on both levels) would quicken the upwelling of smoke. It was expected that exhaustion launched on the level above the fire would have a sucking effect on the fire level. This would have led to a faster spreading of smoke.

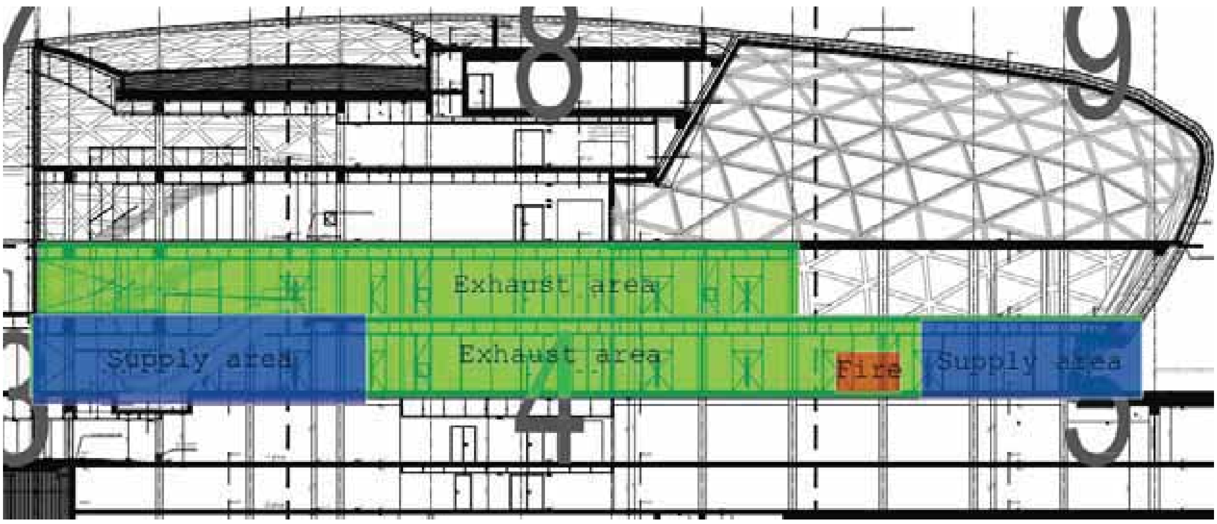


Fig. 29.3 The Smoke control zones

**29.3.3 Analysis of the 'body' and the 'head'**

The area for fresh air inflow was smaller than the one that is ordered by the fire code (in both the 'body' and in the 'head' as well). In the body this surface was only 10, 06 $m^2$ instead of the ordered 30, 34 $m^2$ and in the 'head it was 14, 4 $m^2$ instead of the required 17, 56 $m^2$. In Hungary you must have the permission of the National General Directorate for Disaster Management (Országos Karasztrófavédelmi Főigazgatóság) if you would like to implement a heat and smoke exhaust system that does not suit the requirements of the fire code. In order to acquire such a permission you need to justify that the system you would like to set up can function reasonably.  We examined this operability with the help of computerized fire-simulation. The base data of such a simulation has to be set up in accordance with the authorities. These data are the following: the heat release of the fire, the location of the fire, the smoke-generating ability of the fire, and the duration of the survey.

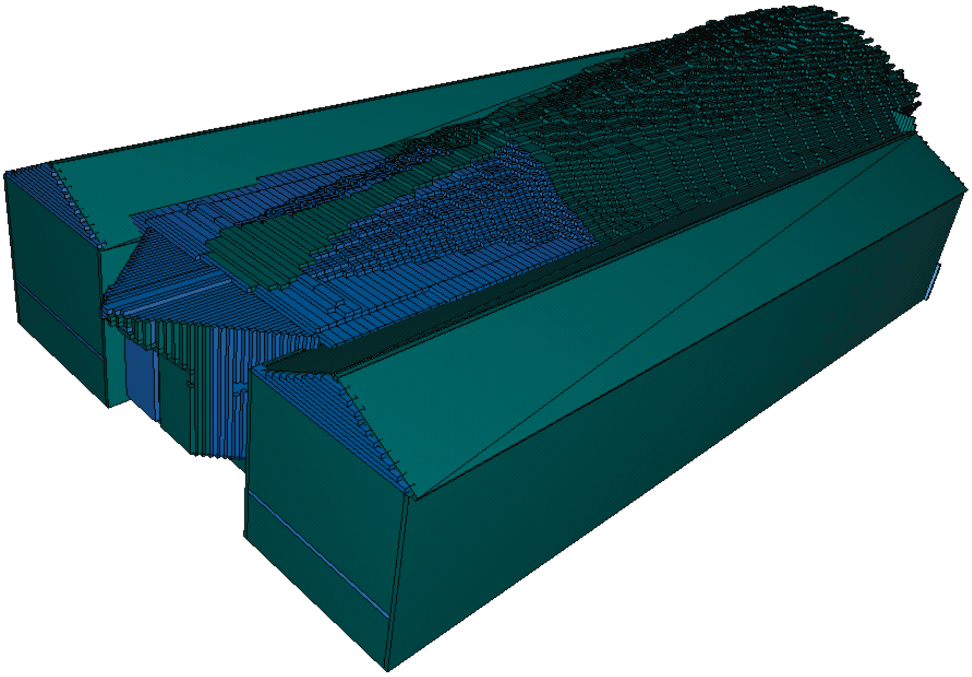

Fig. 29.4 ‘body’ structure from a distance

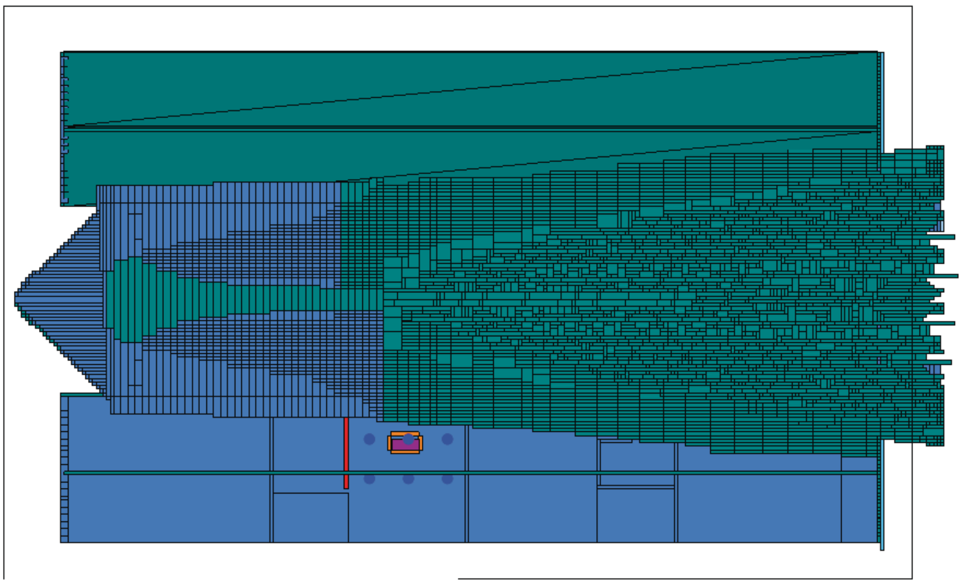

Fig. 29.5 Location of the fire on the first floor of the ‘body’ (the fire is marked by the purple square)

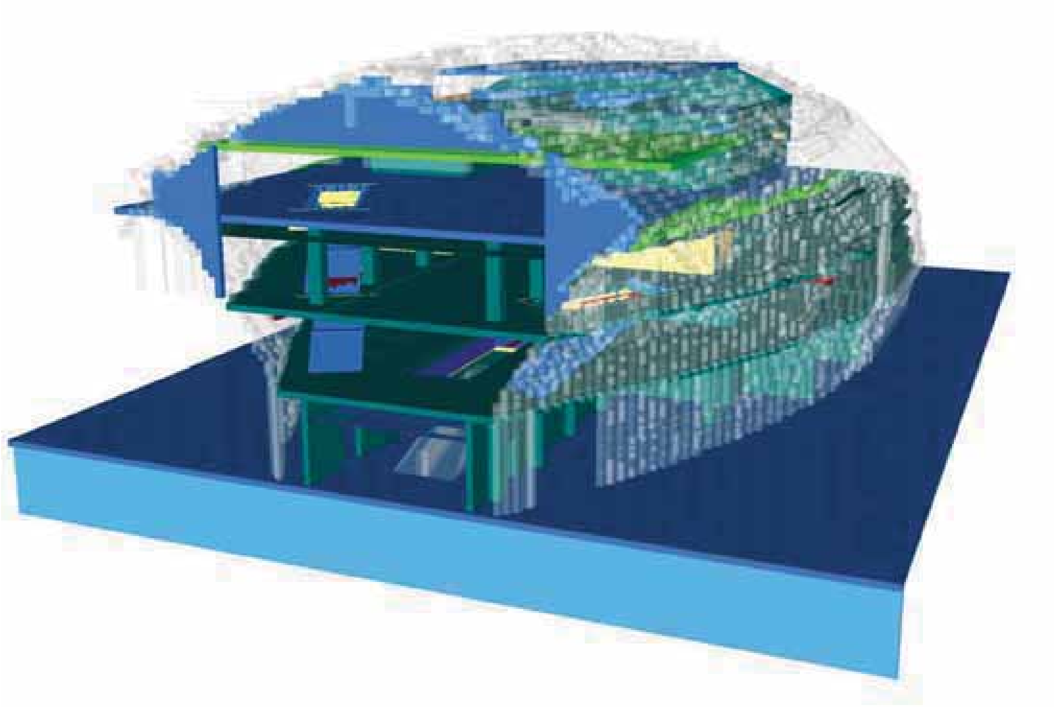

Fig. 29.6 The structure of the 'head'

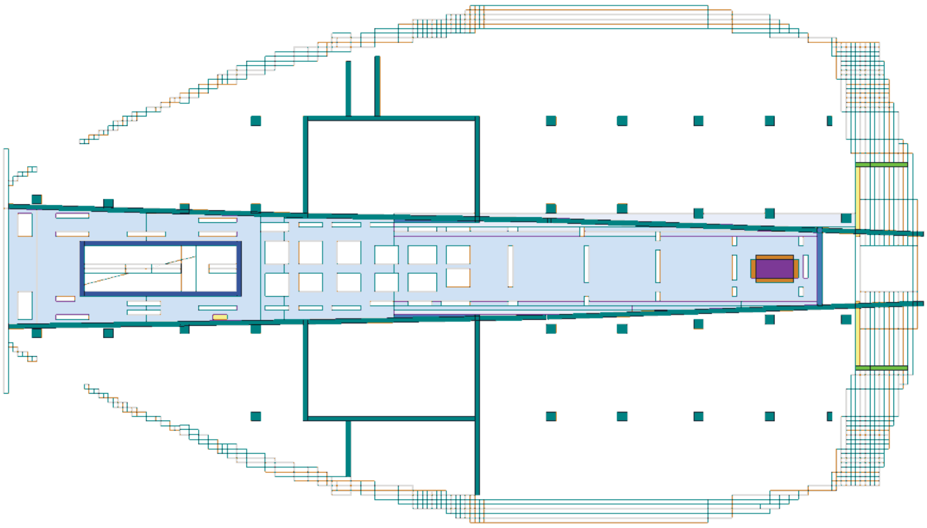

Fig. 29.7 Location of the fire in the 'head' structure on the ground floor (the fire is marked by the purple square)

Smoke could exit the 'body' structure so that no problem was caused and visibility on the escape routes did not worsen. During a simulation authorities in Hungary examine the shift within visibility that is to be calculated with the help of *extinction coefficient.* In Hungary a heat and smoke exhaust system has to provide a 25 meter visibility on the escape routes and a 10 meter visibility for the firemen (at the time of their arrival).

S: *visibility)*[m]

KS: *light occlusion* KS=3; *light emission* KS=8

K:extinction coefficient [1/m]

The distances of visibility can be set up based on this (with the help of the extinction's values).

Tab. 29.1. Distances of visibility for light-occlusive materials (their extinction coefficient differs)

| K extinciós koeff. (extinction *coefficient)* [1/m] | S láthatóság (*visibility*) [m] |
|---|---|
| 0,1 | 30 |
| 0,12 | 25 |
| 0,15 | 20 |
| 0,17 | 17,6 |
| 0,2 | 15 |
| 0,25 | 12 |
| 0,3 | 10 |

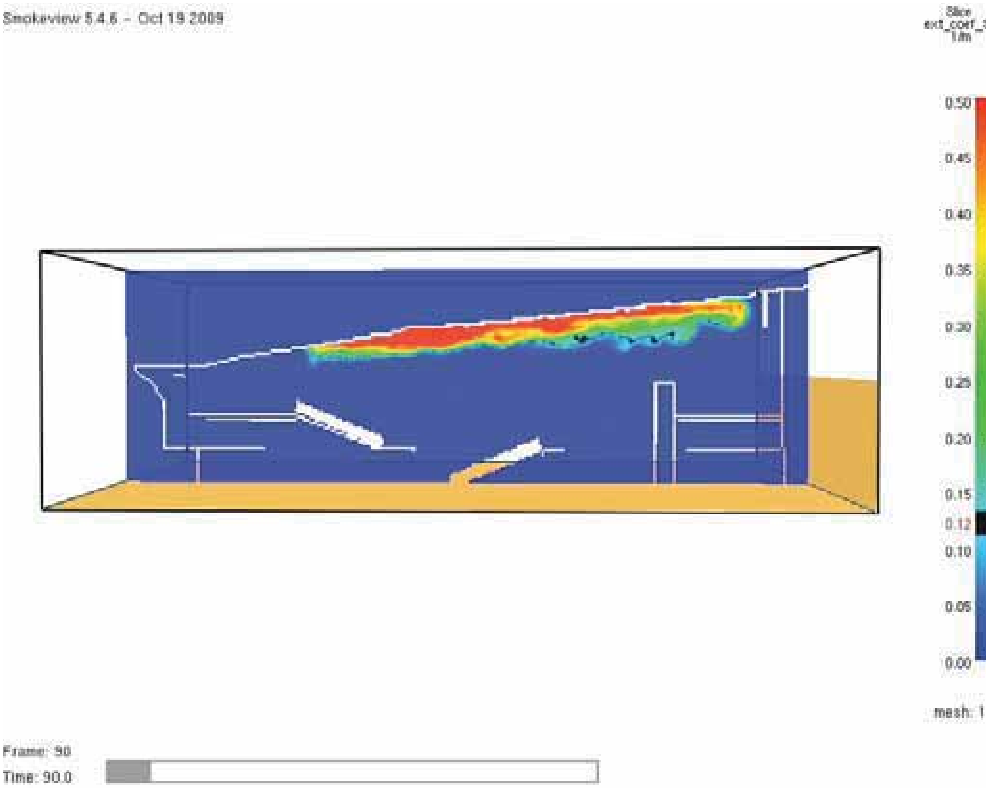


Fig. 29.8 Extinction within the 'body' structure in the 90th second

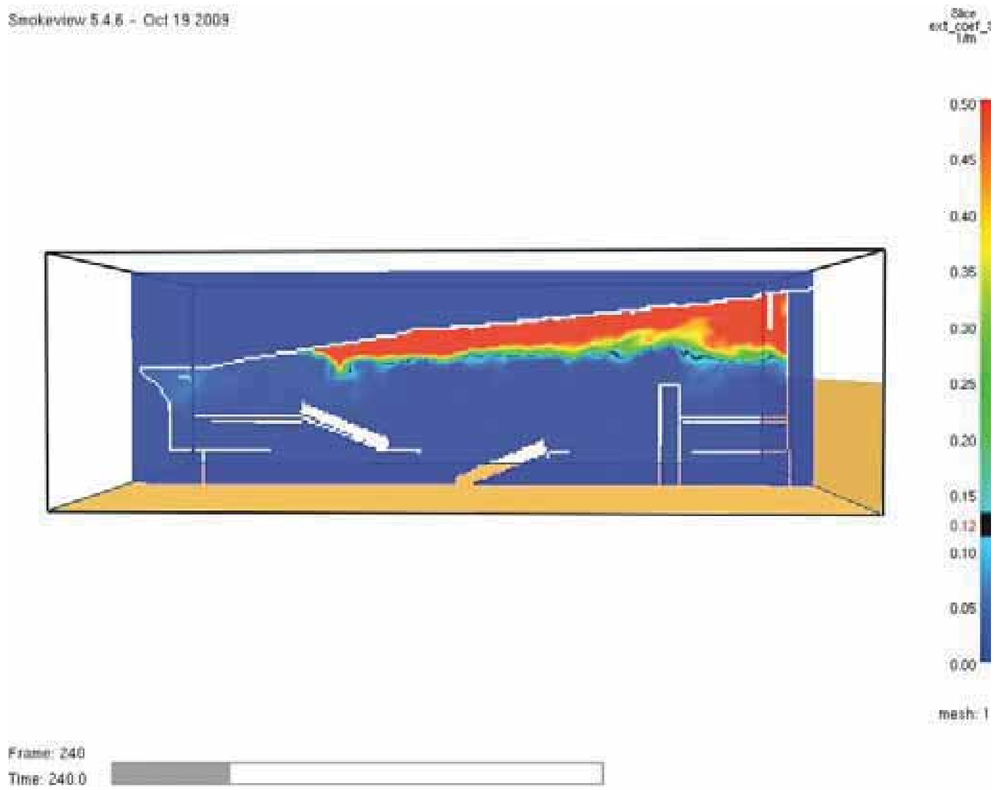


Fig. 29.9 Extinction within the ‘body’ structure in the 240th


Fig. 29.10 Extinction within the ‘head’ structure in the 90th second


Fig. 29.11 Extinction within the 'head' structure in the 240th second

The levels of the 'head' were filled with smoke too fast so we had to come up with another possible solution to the problem.

### 29.3.4 A possible solution to the problem

We refused to increase the surface of air inflow because most probably this was not the factor that caused the faster spread of smoke. Instead, we made two minor modifications in the heat and smoke exhaust system. First, the system only begins its work on the level of the fire and we also built in smoke barriers (35 cm at the stairs).

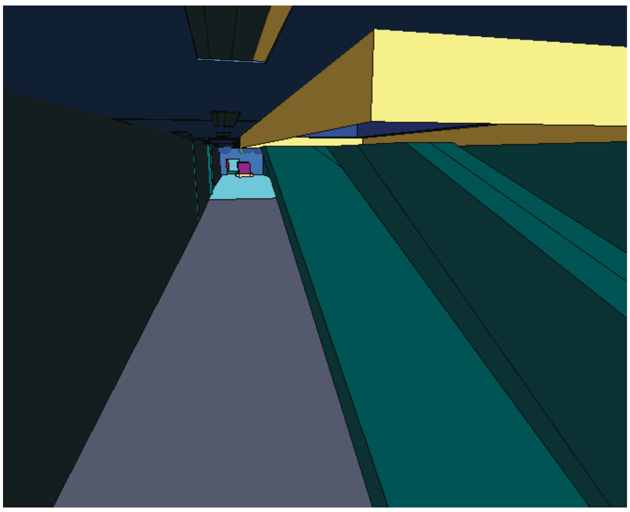

Fig. 29.12 Smoke barriers in the 'head'

### 29.3.5 Analysis of the possible solution

With the help of the smoke barriers the efficiency of the heat and smoke exhaust system increased greatly and the rate at which smoke is spreading has decreased as well.


Fig. 29.13 Extinction in the ‘head’ after its modification in the 90th second


Fig. 29.14 Extinction in the ‘head’ after its modification in the 240th second

### 29.3.6 Results

It can be stated that although the system (of air inflow) does not suit the fire code, this does not cause any difficulties during an evacuation/emergency. We have found that the heat and smoke exhaust system within the 'head' structure was not appropriate (based on the results of the primary test). It was rather probable that the fast spread of the smoke was not caused by the cutting back of the air inflow so we have made two modifications in the system. Namely, the exhaust system only started to work on the level of the fire and a 35 cm smoke barrier was integrated at the floor space (next to the stairs) so that it may slow down the spread of the smoke. Thanks to these modifications it can be claimed that the new system slowed down the rate at which the smoke is spreading and this way the heat and smoke exhaust system has become much more efficient.

Table 3

| According to the model | Suction | Surface provided for air inflow |
|---|---|---|
| 'Body' | 218 700 $m^3/h$ | 10,06 $m^2$ |
| 'Head' | 126 500 $m^3/h$ | 14,40 $m^2$ |

## 29.4 FIRE RESISTANCE INVESTIGATION OF THE STRUCTURE

In Hungary every building structure has to meet a requirement of class of fire risk and critical limit for fire resistance, depending on number of floors and fire resistance degree. The rules are laid in the Hungarian Fire Code (OTSZ, 28/2011. (IX.6.) issued by Ministry for Home Affairs).

The CET building contains partly historical old brickworks and also modern structures. The modern shell structure made from steel and glass has loadbearing and separating function as well and is partial nearly vertical.

In order to find out if fire design of CET building was appropriately fire resistance tests of its modern building structure had to be carried out in the accredited Fire Protection Laboratory of ÉMI Nonprofit Ltd.

### 29.4.1 Walls

The vertical part of the shell structure was investigated as a wall in the vertical testing furnace.It was loaded and the steel frame was coated with fire-proof painting. The examination of critical limit for fire resistance of the structure was carried out in according to the regulations of the MSZ EN 13651: 2000 standard.

Fig. 29.15 Test model of the wall construction before the fire resistance test

Fig. 29.16 Test model of the wall construction during the fire resistance test

Fig. 29.17 Tested wall model after the fire resistance test

(left – unexposed side, right – fire exposed side)

### 29.4.2 Ceiling

The horizontal part of the shell structure was investigated as a ceiling in the horizontal testing furnace. It was loaded and the steel frame was coated with fire-proof painting and its flexibility was under scrutiny as well. The examination of critical limit for the fire resistance of the structure was carried out in accordance with the regulations of the MSZ EN 13652: 2000 standard.

Fig. 29.18 Test model of the ceiling before the fire resistance test

Fig. 29.19 Test model of the ceiling during the fire resistance test

Fig. 29.20 Tested ceiling model after the fire resistance test

## 29.5 SUMMARY

Based on the information above, it can be claimed that those buildings which have a special structure and implementation require studies concerning their fire resistance (laboratory study) and fire simulations (FDS) as well. If a structure has a critical fire resistance value that is lower than the required value, then additional protective steps are required (such as integrating sprinkler heads or a smoke control system). Such protective arrangements in the case of CET are the coating of the steel frames (that have the required cross-section) with fire-proof paint and the fact that the filling elements are made of fire-proof glass and solid, insulated materials. It can be stated that CET's heat and smoke exhaust system is safer and more economical than what is prescribed by the laws. In Hungary one is only allowed to implement a heat and smoke exhaust system that is different from the one prescribed by the law, after proving that the system is able to function appropriately. Computational simulation is one of such justification methods (but it should be noted that the base data is always set by the authorities).

Acknowledgement

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-Architect designer: Kas Oosterhuis

-General main contractor: WHB Építő Kft.

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# 30 EXAMINATION, ASSESSMENT AND REPAIR OF FIRE DAMAGED RC STRUCTURE OF „REFINERY-OKTA" IN SKOPJE

Summary

In November 2008 a part of the structure of "Refinery–Okta" in Skopje, Macedonia, was in fire and the last three spans were completely burned. According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure. For realization of the repair project of the RC structure, nonlinear and transient thermal analysis, nonlinear stress-strain analysis of the frame structure and experimental determination of the residual concrete strength after action of fire were recommended. Results obtained by this analysis are presented in this paper. The global repair recommendations are given, also.

## 30.1 INTRODUCTION

Three years ago a part of the structure of "Refinery-Okta" in Skopje, used for primary processing of crude oil, was in fire. The burning process was supported by the crude oil and gasoline. The fire started in the second span of the open structure, but quickly spread to the first three spans. Fire duration, according to the witnesses, was about one our, but the caused effect indicates that it lasted longer and that temperature in the fire section was up to 1100$^{o}$C. Fire was extinguished by using water and foam.

## 30.2 EXAMINATION AND ASSESSMENT OF FIRE DAMAGED RC STRUCTURE

### 30.2.1 Basic data about structure

The object was built in 1978 and the technical project documentation was done by a design bureau from Moskva, Rusia. The load-bearing structure is classic skeletal reinforced concrete structure (Fig. 30.1, 30.4). Special cooling systems are placed on the slab. The reinforced concrete frames are placed on 6.0m. Basic frame elements are:

- columns S1, dimensions 60 x 80 cm,
- beams G1, dimensions 55 x 100 cm,
- crane beams G2, dimensions 35 x 70 cm.

The beams that support the reinforced concrete full slab (8.0 cm thick), are placed in longitudinal direction:

- beams B1(at the edge), dimensions 30 x 60 cm,
- beams B2, dimensions 45 x 60 cm,
- beams B3, dimensions 30 x 60 cm.

Reinforcement details were available from the technical project documentation. The design value of concrete compressive strength, for all elements, is $f_c$ = 30Mpa. All columns and beams have ribbed bar reinforcement, type A III (according to our regulation RA 400-500). Stirrups are from mild reinforcement Ø8, type A I (according to our regulation GA 240-360). For the slabs reinforcement type AI and AIII is used.
Data on the design loads, as well as static and dynamic calculations, were not available.

### 30.2.2 Visual inspection of structure

Based on the data gathered through detailed visual survey of the bearing structural elements, the following characteristic damages of beams and slabs were recorded: change of concrete color; fissures and cracks inside the concrete mass; cracks along main reinforcement, crushing of concrete and falling off of concrete parts along the edges of linear elements up to the reinforcement. White color of the surface concrete layers was caused by the chemical reaction between the water, used for extinguishing the fire, and the dehydrated carbonate aggregate (Fig. 30.1, 30.2). This phenomenon is followed by expansion of concrete mass up to 44% and causes cracks in concrete mass (Bazant & Kaplan, 1996).

Due to the openness of the space and the continuous supply of oxygen and oil, the flames spread high under the slab and the columns were exposed to lower temperatures. These elements were not visually damaged, cracks were not recorded and the concrete color was rose-red, that indicates temperatures less then 600$^{o}$C.

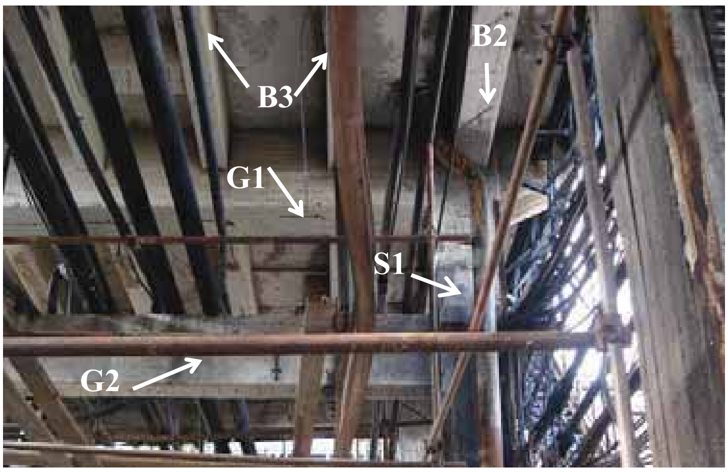


Fig. 30.1 Disposition of the beams in the second spam and change of concrete color due to chemical reaction

### 30.2.3 Experimental determination of the residual concrete strength

Temperature over 400°C causes reduction of the compressive strength and other mechanical properties of concrete and this process is irreversible (the strength of concrete does not recover in the cooling phase). The mechanical properties of hot welded steel (reinforcing bars) decrease as well, but in the cooling phase they increase again. According to these statements and for realization of the repair project of the RC structure, experimental and numerical determination of the residual concrete strength was recommended. Based on previous Schmidt hammer testing results, the locations of the eight concrete specimens, taken only from lateral side of the RC beams, were defined (Fig. 30.2).

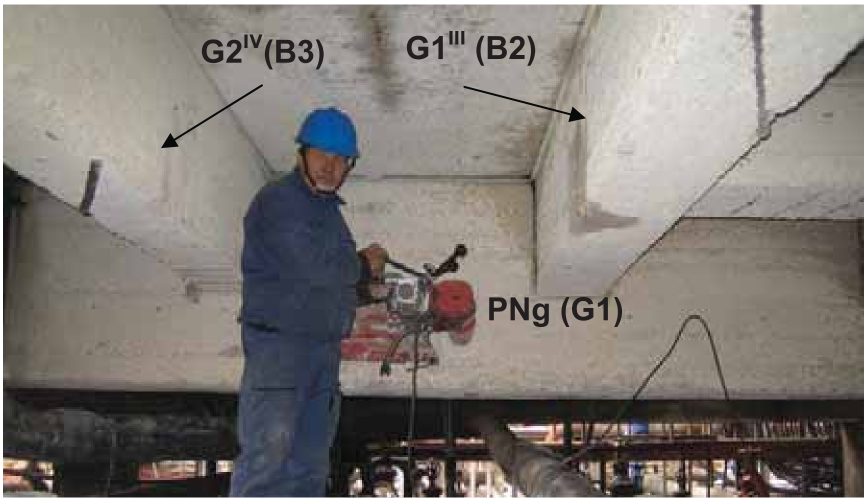


Fig. 30.2 Locations where the concrete specimens were taken from the beams of the second spam and change of concrete color due to chemical reaction

The concrete specimens were tested at the Testing laboratory of the Civil Engineering Faculty in Skopje. Before testing all the specimens were divided in two slices. The deteriorated (burned) slices had small height (3-6 cm) and rough surface (Fig. 30.3). Resulting average value for the residual concrete strength of burned slices was $f_{c,r}$ = 9.24 Mpa, (Cvetkovska & Lazarov, 2008). Lower values were expected for the concrete layers at the bottom of the beams (directly exposed to the flames) but, because of the position of the reinforcement, specimens were not taken from this location. This statement and the testing results were confirmed by the numerical thermal analysis of the fired beams. Hammer testing, testing results of the concrete specimens taken from locations that were not fired, as well as test results of the unburned slices, confirmed that the compressive strength of the carbonate aggregate concrete before the action of fire was $f_{co}$= 30 MPa.


Fig. 30.3 Concrete specimens before testing

**30.2.4 Numerical determination of the residual concrete strength**

Thermal response of the fired structure (Fig. 30.4) has been investigated analytically, too. Elements geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio and defined fire scenario were taken into account while the nonlinear and transient temperature field and the concrete strength reduction in the cross section of the elements exposed to fire were determined.

The computer program FIRE, (Cvetkovska, 2002), was used to solve this problem and the following assumptions were made:

- Fire was modeled by a single valued gas temperature history and in this case ISO 834 fire model was used. According to data gathered in situ it was assumed that the maximum fire temperature of 1100$^{o}$C was reached at the moment t= 1.5hours and after that the cooling period started. This temperature was reached only in the zones under the slab, but in the lower zones it was approximately 50% less than the maximum one.
- Temperature dependent material properties were known (recommended in EC2).
- The fire boundary conditions were modelled in terms of both convective and radiating heat transfer mechanisms.
- The easy heat penetration, after cracks had appeared, or some parts of the cross section had crushed, was neglected.

The results of the nonlinear thermal analysis are presented by graphs. The isotherms in the cross section of the beam B2, at moment t = 1.5 hour, are presented on Fig. 30.5a. The beam was fired only from the bottom and lateral sides but, because the space was open, at the opposite side (over the slab) the temperature was proportional to the temperature under the slab (approximately 25% of the temperature reached under the slab) and the beam was heated from both sides, but not with the same intensity.

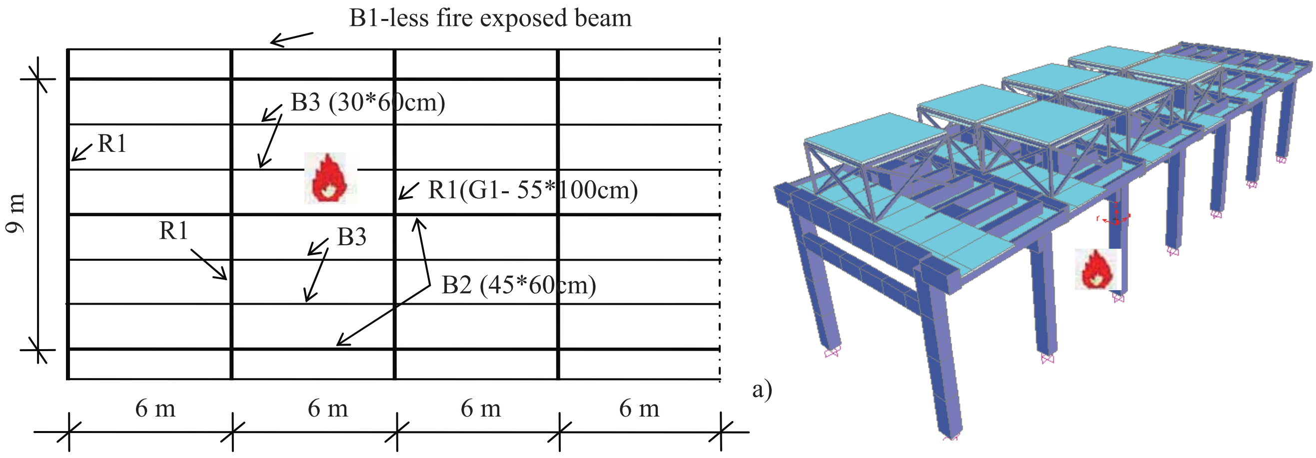


Fig. 30.4 a) Schematic presentation of the fired structure

b) Model of structure in SAP2000

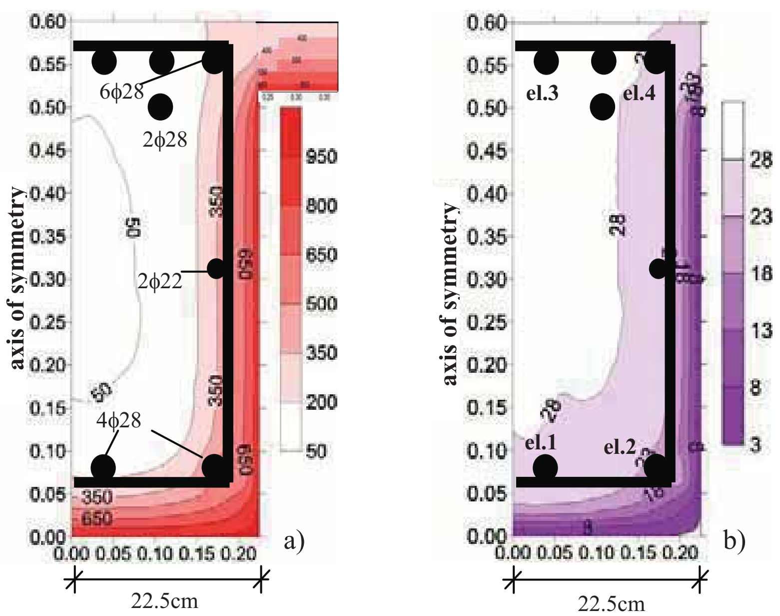


Fig. 30.5 a) Isotherms in the cross section of the beam B2 (t=1.5hours),

b) residual concrete strength after cooling phase

The residual concrete strength after cooling period is presented on Figure 5b. Calculation results indicate that on the side of the fire, in 3-4 cm thick layer, which is 25 - 30% of the cross section of the beam, the strength reduction is significant and the residual strength of concrete is 10 Mpa in average. These results correspond well with experimental results obtained by laboratory testing of specimens taken from the same RC beam. In the cross section core the strength of concrete is not reduced.

While the elements were built, the stirrups were not well tied for the reinforcing bars and almost in all beams there are thick concrete layers between the stirrups and the bars. The bars are placed high in the cross section and the concrete cover thickness from the bottom side is 6-7 cm. This has adversely effect from the aspect of the internal lever arm and the bearing capacity of the beams, but in case of fire it helps the temperatures of the reinforcing bars to be lower than in case when the concrete cover thickness is only 2-3 cm. The time dependant temperatures of the reinforcing bars of the beam B2 are presented on Fig. 30.6.

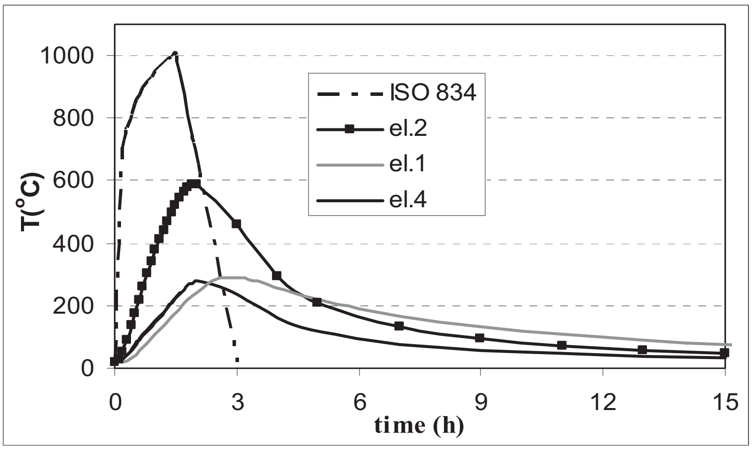


Fig. 30.6  Time dependent temperatures of the reinforcing bars of the beam B2 which is exposed to fire from three sides

The beams B3 and the beams G1 (Fig. 30.1), that are part of the reinforced concrete frames and supports beams B2 and B3, were exposed to fire from three sides, too. Calculation results indicate similar temperature distribution as for the beams B2.  In 3-4 cm thick layer, which is near 20% of the cross section of the beam G1, the residual concrete strength is 10 Mpa in average.

The beams G2, that support the crane, were fire exposed from all sides, symmetrical temperature field in the cross section of the beams was generated and near 30% of the cross section has got concrete strength less then 20 Mpa (10 Mpa in average). Results are presented on Fig. 30.7 a,b.

The time dependant temperatures of the reinforcing bars of the beam G2 are presented on Fig.30.8.

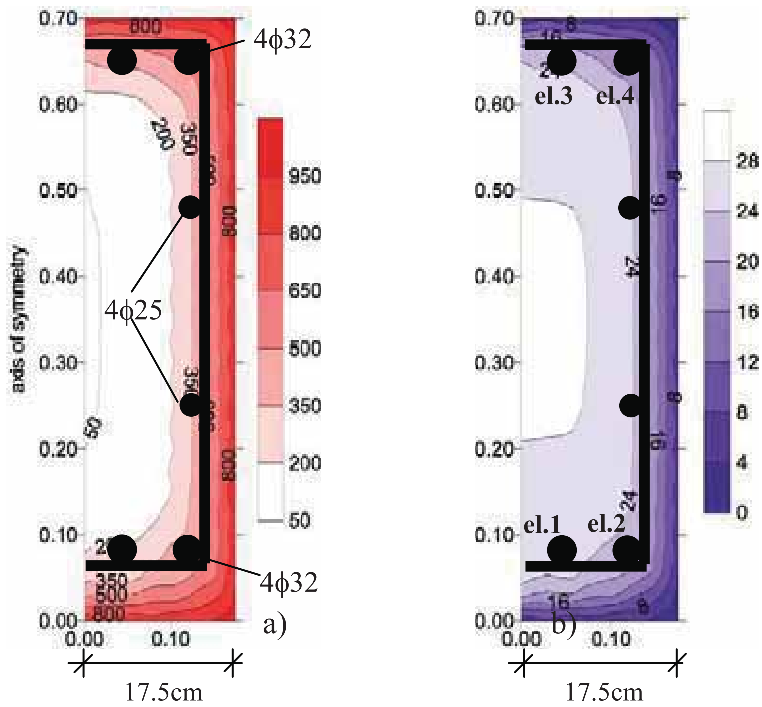


Fig. 30.7 a) Isotherms in the cross section of the beam G2 (t=1.5hours),

b) residual concrete strength after cooling phase

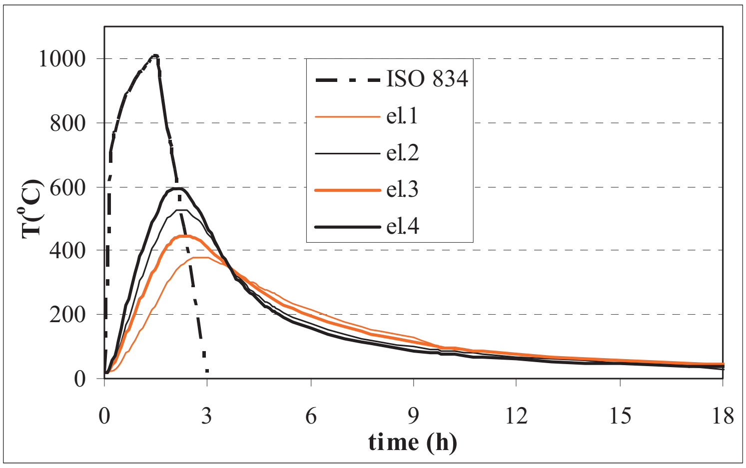


Fig. 30.8 Time dependent temperatures of the reinforcing bars of the beam B2 which is exposed to fire from three sides

## 30.3 REPAIR OF FIRE DAMAGED REINFORCED CONCRETE STRUCTURE

### 30.3.1 Nonlinear stress-strain analysis of the fire exposed reinforced concrete structure

Data on the design loads, as well as static and dynamic calculations of the bearing structure, were not available from the project documentation. For the needs of the Repair project additional calculations for normal temperatures (without fire action) were conducted. For that purpose the program SAP2000 was used (Fig. 30.4b). There was lack of data on the intensity of the dynamic loads of the cooling system placed on the slab and they were not taken into account, but the structure was controlled on seismic loads. The conclusion confirmed that the bearing structure was well designed and there were sufficient reserves for adoption of the dynamic impact of the cooling system.

The response of the bearing structure, while exposed to extremely high temperatures during fire action, as well as in the cooling phase, was predicted by the program FIRE, (Cvetkovska, 2002). This program carries out the nonlinear transient heat flow analysis (modulus FIRE-T) and nonlinear stress-strain response associated with fire (modulus FIRE-S). The solution technique used in FIRE is a finite element method coupled with time step integration. The program accounts for: dimensional changes caused by temperature differences, changes in mechanical properties of materials with changes in temperature, degradation of sections by cracking and/or crushing and acceleration of shrinkage and creep with an increase of temperature. The used analysis procedure does not account for the effects of large displacements on equilibrium equations.

Fig. 30.9a presents discretization of beams B2 and B3 (supported by frames R1), Fig. 30.9b presents discretization of frame R1 (axial symmetry is used).

Fig. 30.9c presents discretization of the cross section of the elements (isoparametric finite elements with four node are used). Period of 30 hours was analyzed. The cooling phase was important for defining the residual bearing capacity of the structure.

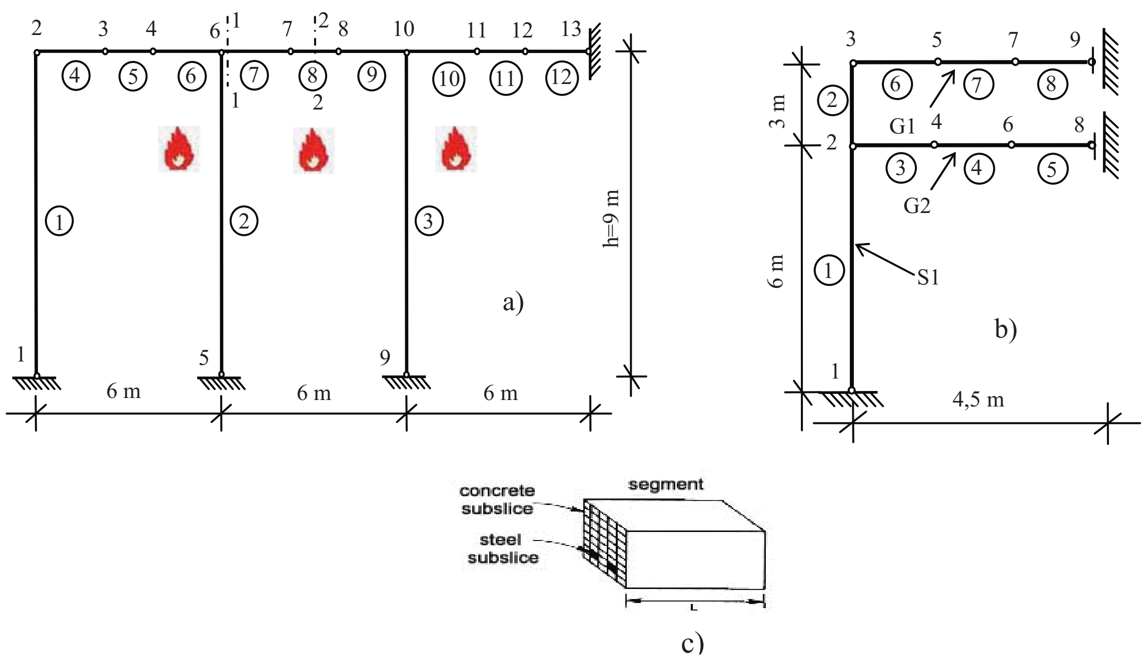


Fig. 30.9 Schematic presentation of the fired structure and discretization of the elements:

a) longitudinal beams B2 and B3, b) main frame R1, c) discretization of the cross section

The columns S1 (60 × 80 cm) are symmetrically reinforced with $20\phi32$ ($f_y$ (20°C) = 400 Mpa). Before the action of fire the compressive stress in concrete core was 1.2 Mpa, that was only 4% of $f_c$ (20°C), and the stresses in all reinforcing bars were 64Mpa, that was only 16% of $f_y$ (20°C). Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers. This redistribution was highly expressed at the moment when the fire was extreme (t = 1.5 hours), but it didn't cause cracking, or crashing of concrete, nor yielding of the reinforcing bars. Only the upper parts of the columns were exposed to fire with the same intensity as beams and slabs were, therefore critical stresses occurred only in the cross sections close to the slabs, but the extreme values were +363 Mpa, or 90% of $f_y(T)$, at the inside and -250 Mpa, or 63% of $f_y(T)$, at the outside of the column cross section. After the cooling phase the stresses were back to 30% of yielding strength of the bars for room temperature. Therefore the repair of the RC columns was not recommended.

The bottom and lateral sides of the beams B2, B3 and G1 were directly exposed to fire. They became hotter than the top sides and tended to expand more. This differential heating caused the ends of the elements to tend to lift from the supports thus increasing the reactions. This action resulted in a redistribution of the bending moments. The negative moments increased, while the positive moments decreased and tended to become negative. After time t = 0.5 h, the negative moments began to decrease again.

The concrete, directly exposed to fire, developed large compression stresses due to thermal gradients. At the ends of the beams fire had the same effect as the uniform load did, therefore the concrete at the bottom side of the cross section crushed, while concrete at the top side of the cross section cracked. The effect was opposite at the middle of the span.

The thick concrete layers (6-7 cm) on the bottom side of the beams protected reinforcement from high temperatures, but after a time the bars which were close to the fire became hotter, and opposite (Fig. 30. 6, 8). The increase of negative moments at the ends of the beams was accommodated, but the redistribution that occurred was sufficient to cause yielding of the top bars which were in the corners. In the same cross section, the reinforcing bars which were on the side of the fire were all the time in compression by the action of the fire and the negative bending moment (el.2 on Fig. 30.10, 12). The yield strength was reduced due to the high temperatures, hence the reinforcement started to yield very soon. Large plastic deformations occurred at the end of the heating period, so during the cooling phase the reinforcement changed the sign and residual stresses in tension occurred. The residual tension stresses were significant for bottom bars placed at corners of beams B2 and B3 and their values were up to 90% of the yielding strength at room temperature (Fig. 30.10). Beams G1 were in a better situation. The residual tension stresses were not higher then 50% of the yielding strength at room temperature (Fig. 30.12), therefore the repair of the beams G1 was not recommended.

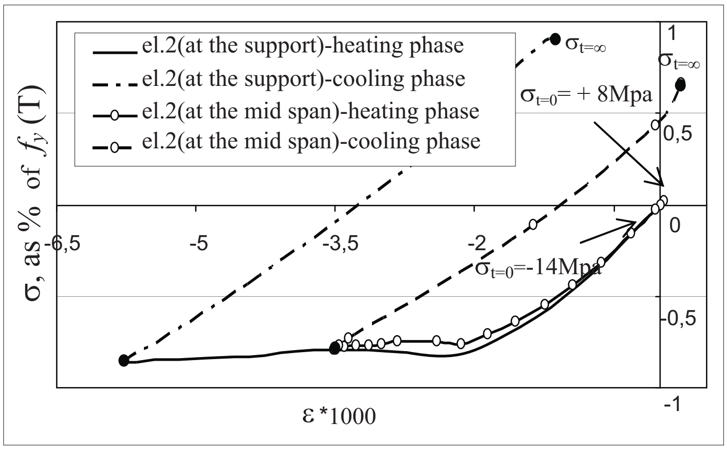


Fig. 30.10 Time dependent stresses of reinforcement no.2 of beam B2, for cross section at support and mid span, as a percent of yielding strength at corresponding temperature

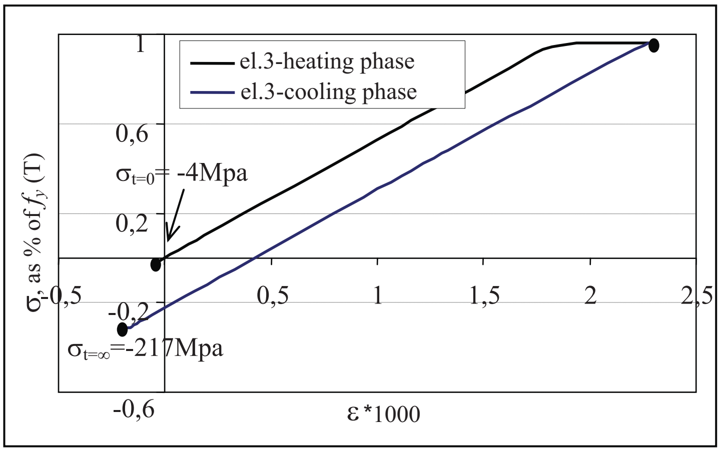


Fig. 30.11 Time dependent stresses of reinforcement no.3 of beam B2, for cross section at mid span, as a percentage of yielding strength at corresponding temperature

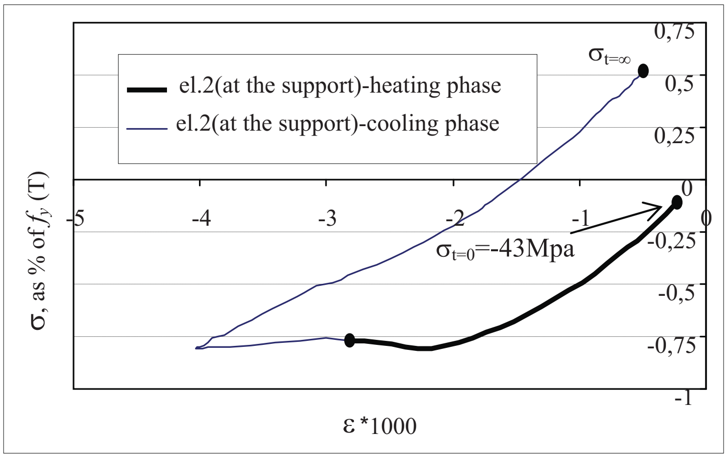


Fig. 30.12 Time dependent stresses of reinforcing bar no.2 of beam G1, for cross section at support, as a percent of yielding strength at corresponding temperature.

The stress-strain diagram for the mid span top reinforcement (temperatures less then 200°C) is as for room temperatures (Fig. 30.11).

Crane beams G2 were exposed to fire from all sides, it caused symmetrical cross section temperature field (Fig. 30.7) and temperature induced stresses were lower than for the other beams. During the cooling period the stresses in reinforcement changed the sign too, but the residual stresses were less then 30% of the yielding strength for steel at room temperatures. Therefore the repair of the beams G2 was not recommended.

### 30.3.2 Suggested repair of fire damaged structural elements

The aim of the repair suggested in this project was to obtain nearly the same strength, stiffness, deformability and ductility of the repaired elements and their cross sections as they had before the fire action.

The first step of the suggested repair of beams B2 and B3 (in longitudinal direction) was elimination of the deteriorated (burned) concrete layers up to the reinforcement (Fig. 30.13). For that purpose 3-4 cm thick layers, from the lateral side, and 6-7 cm thick layers, from the bottom side, were removed. Before casting the opened sections were cleaned with running water.

Fig. 30.13 Elimination of the deteriorated (burned) concrete layers up to the main reinforcement

The yield reinforcement at the corners of the beams, with total area of 12 $cm^2$, was “covered” by additional reinforcement (4Ø20, RA 400/500), placed under the existing stirrups (Fig. 30.14). The additional open stirrups Ø8/20 cm (GA 240/360) were welded to the existing stirrups. Welding between the new ribbed reinforcement and existing smooth stirrups was not recommended for two reasons: the additional reinforcement had higher yield strength and the existing smooth stirrups had residual plastic deformations. The jacketing with new 3cm thick concrete layer was made with sprayed concrete MB 30.

As it was shown before, the only suggested repair of beams G1 and G2 (RC frames R1) was in replacing the "burned" concrete layers with new one.

The results obtained by the thermal analysis of the RC slabs and the detailed visual survey in-situ led to a conclusion that the concrete strength was reduced only from the bottom side of the slabs where concrete is in tension and does not influence upon the bearing capacity of the slabs. The fact that the slab's span was only 1.5 m, additionally confirmed that the suggested repair of the slabs should be in replacing the deteriorated concrete layers by new one.

Fig. 30.14 Repair of the beam B2 at the second spam.

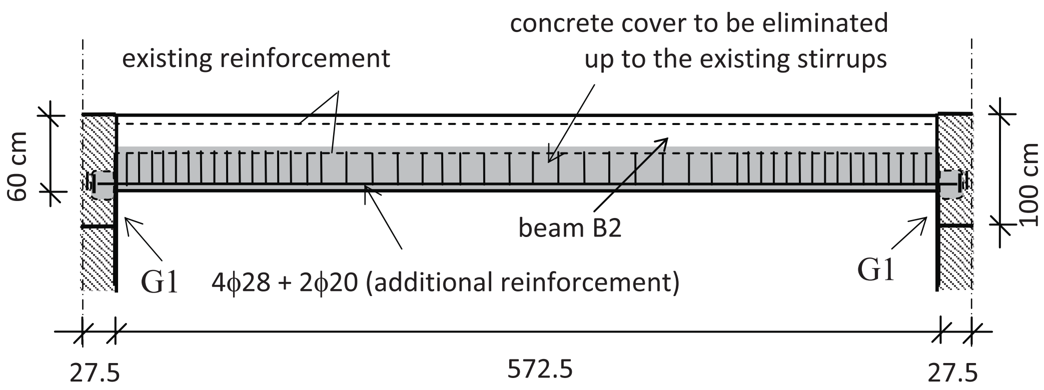


Fig. 30.15 Repair details of beam B2.

Fig. 30.16 Additional stirrups of beam B2, welded for the existing one.

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# 31 EXAMINATION, ASSESSMENT AND REPAIR OF RC STRUCTURE OF BUILDING DAMAGED IN FIRE

Summary

A twenty storey building structure was in fire and two apartments at the seventh and eighth floor were completely burned. According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure. Based on the data gathered through detailed visual survey of the bearing structural elements, nonlinear and transient thermal analysis, nonlinear stress-strain analysis of the frame structures and experimental determination of the residual concrete strength after action of fire were recommended. Results obtained by these analysis show that during the fire action, as well as in the cooling period, the strength and stiffness of structural elements were continually reduced and adequate repair of the damaged elements has to be made.

## 31.1 INTRODUCTION

In February 2005 a twenty storey building structure on “Nikola Parapunov str. no. 3, Skopje, Macedonia” was in fire and two apartments at the seventh and eighth floor were completely burned. Primary, the fire was caused by gas explosion, but the synthetic materials in the apartment were additional fire load, so very high temperatures were reached and the fire time was more than four hours.

According to the damages recorded in situ, it was found out that fire caused severe damage to the reinforced concrete bearing structure, while the interior and the installations were completely destroyed. Based on the data collected through detailed visual survey of the bearing structural elements (columns, beams, slabs and RC walls), the following characteristic damages were recorded:

- change of concrete color (red, grey-yellow, yellow, Fig. 31.1)
- fissures and cracks inside the concrete mass (Fig. 31.1d)
- cracks along main reinforcement in columns, beams and slabs (Fig. 31.2)
- crushing of concrete and falling off of concrete parts along the edges of linear elements up to the reinforcement (Fig. 31.2d)


Fig. 31.1 Change of concrete color and characteristic damages of RC elements, recorded in situ

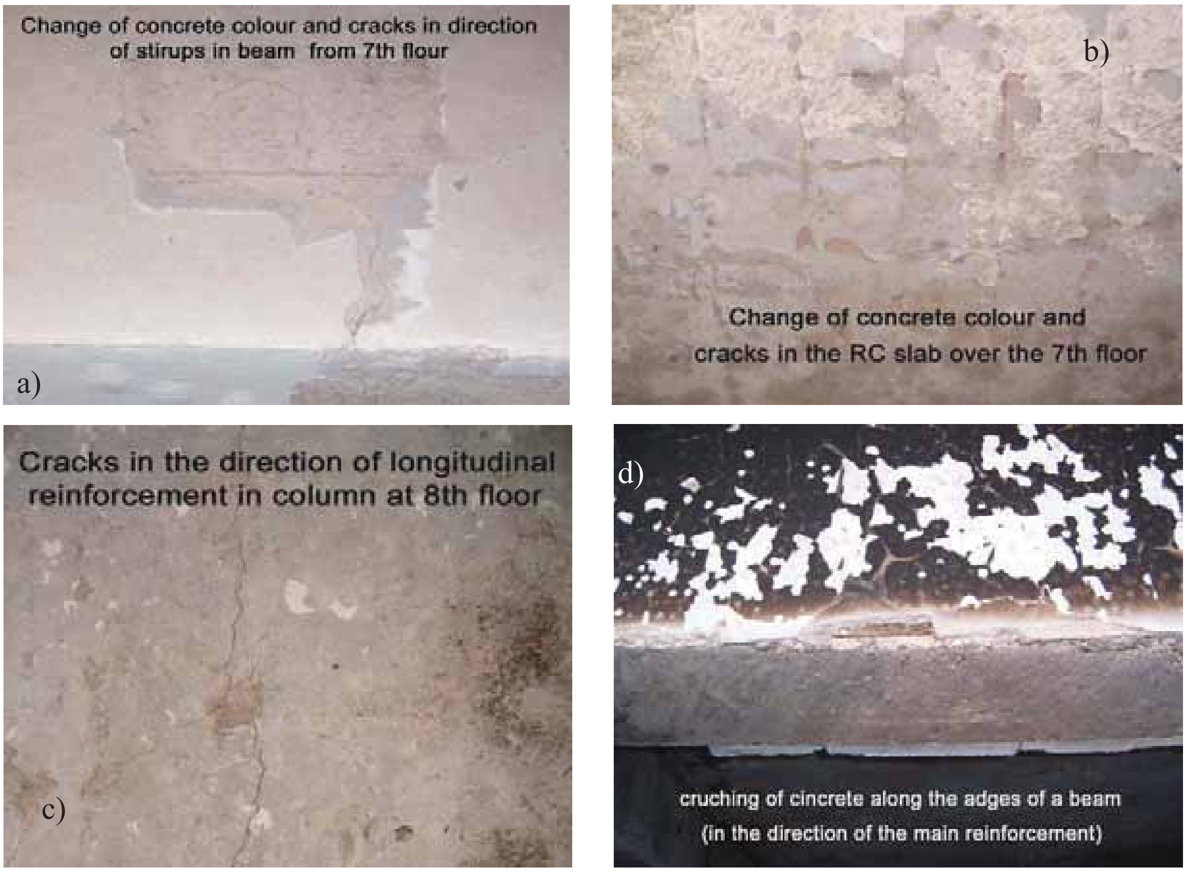


Fig. 31.2 Cracks along main reinforcement in columns, beams and slabs, recorded in situ

According to that situation, the following steps were recommended:

- Control of the element geometry; concrete cover thickness; type of aggregate; compression strength of concrete and the number, type and diameter of the built-in reinforcement, according to the design, as well as control of the loads used in the design calculations.
- Experimental determination of the residual concrete strength after action of fire.
- Nonlinear transient thermal analysis and nonlinear stress-strain analysis of the frame structures exposed to fire and assessment of the degree of damage, according to the results obtained by this analysis.

## 31.2 EXPERIMENTAL AND NUMERICAL DETERMINATIO OF THE RESIDUAL CONCRETE STRENGTH

The possibility for adequate repair of the damaged elements and the measures that have to be done in that case, directly depend on the level of the damages caused during the fire action, as well as in the cooling period. One of the most important factors that directly influence the repairing possibility is the residual compressive strength of concrete. The mechanical properties of the reinforcement decrease as well, but in the cooling phase they increase again.

Temperature over $400^0$C causes irreversible reduction of the compressive strength and other mechanical properties of concrete. The compressive strength of concrete does not recover in the cooling phase because of initial degradation and chemical decomposition of the cement past. The residual compressive strength of concrete should be determinate by laboratory tests of specimens taken from the RC elements exposed to fire, but very often this procedure is impractical because additional destruction of the damaged elements is not advisable. Because of the position of the reinforcement in the surface layers of the cross section, taking the specimens from columns and beams is more complicated and is not advisable. In such cases the problem can be solved by using a numerical procedure, based on the nonlinear transient heat flow analysis and the nonlinear stress-strain analysis. When the residual compressive concrete strength is numerically determined, it’s value is assumed to be the same as the value that corresponds to the maximum concrete temperature.

### 31.2.1 Experimental determination of the residual concrete strength

The experimental testing of the residual concrete strength after the fire action was the first step of the suggested measures and was done by the Institute for Materials Testing and Development of New Technologies “Skopje”-Skopje. According to the previous Schmidt hammer testing results, the locations of the eight concrete specimens, taken only from RC walls and RC slabs, were defined. Hammer testing of concrete elements from apartments that were not fired confirmed that the compressive strength of the concrete before the action of fire was between $f_c$=30MPa and $f_c$=40MPa.

The concrete specimens taken from the RC walls (B8, V7, G7, D7, E7 in Tab. 31.1) were exposed to fire from one side. The corresponding surface layers (3-5cm thick) had changed the color (red, grey-yellow, yellow) and were more deteriorated than the inner layers (fig.3a). The RC slabs over the 7$^{th}$ floor were fire exposed from both sides, but they were covered with 1cm thermal isolation and 4cm lean concrete (Fig. 31.1c and Fig. 31.3b), that directly influenced upon the cross section temperature field, therefore the specimens taken from these slabs (T2, MS, CII) were deteriorated only from one side (the bottom side), too.

Fig. 31.3 Deterioration of surface layers of specimens

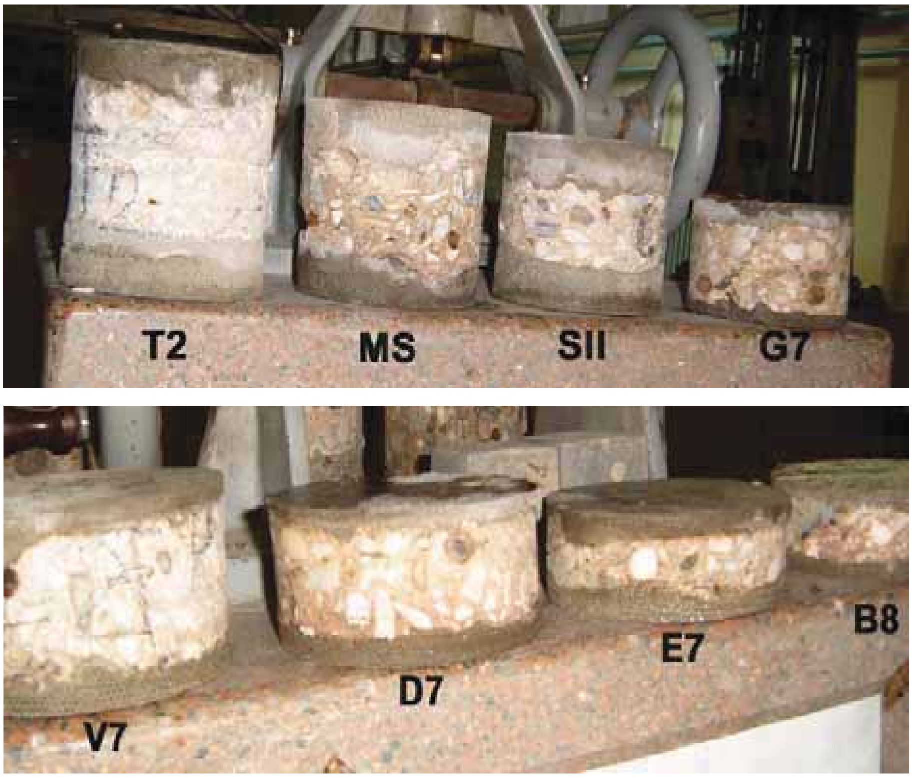

Fig. 31.4 Deteriorated concrete specimens, prepared for testing

Before testing all the specimens were divided in two slices. The deteriorated (burned) slices had small height (3-6cm) and rough surface therefore they were specially prepared by adding plaster layers (fig.4). In that case the measured values for the compressive concrete strength were reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens. Test results are presented in Tab. 31.1.

Tab. 31.1 Concrete strength testing results

| No. | Position | Dimensions of kerns (cm) | | | density ($kg/m^3$) | Test load (KN) | Concrete compressive strength (MPa) | | Age of Concrete (years) | Design concrete strength according to MKS U.M1.048 |
|---|---|---|---|---|---|---|---|---|---|---|
| | | total H | h (testing) | D | | | cylinder | reduced to cube 20/20/20 | | |
| 1 | RC wall (B8) | 15 | 9.9 | 9.9 | 2214 | 200 | **26.0** | 26.5 | 22 | **20.0** |
| | (after fire expose) | | 3.5 | | 2150 | 390 | **15.8*** | 16.1 | | **12.0** |
| 2 | RC slab (T2) | 10 +7cm plaster | 10 | 9.9 | 2300 | 215 | **28.0** | 28.6 | 22 | **21.4** |
| | (after fire expose) | | / | | / | / | **/** | / | | **/** |
| 3 | RC slab (MS) | 18 | 10.3 | 9.9 | 2340 | 260 | **33.8** | 34.5 | 22 | **26.0** |
| | (after fire expose) | | 6.5 | | 2310 | 145 | **12.5*** | 12.8 | | **9.5** |
| 4 | RC slab (SII) | 18 | 9.6 | 9.9 | 2315 | 295 | **38.4** | 39.1 | 22 | **29.4** |
| | (after fire expose) | | 4 | | 2269 | 160 | **13.0*** | 13.3 | | **10.0** |
| 5 | RC wall (V7) | 25 | 9.9 | 9.9 | 2117 | 272 | **35.4** | 36.1 | 22 | **27.0** |
| | (after fire expose) | | 5 | | 2138 | 265 | **14.5*** | 14.8 | | **11.0** |
| 6 | RC wall (G7) | 19.4 | 10 | 9.9 | 2179 | 195 | **25.4** | 25.9 | 22 | **20.0** |
| | (after fire expose) | | 3.5 | | 2168 | 290 | **14.5*** | 14.5 | | **11.0** |
| 7 | RC wall (D7) | 18 | 10 | 9.9 | 2212 | 236 | **30.7** | 31.3 | 22 | **24.0** |
| | (after fire expose) | | 3.5 | | 2168 | 290 | **14.5*** | 14.8 | | **11.0** |
| 8 | RC wall (E7) | 16 | 10 | 9.9 | 2259 | 246 | **31.9** | 32.6 | 22 | **25.0** |
| | (after fire expose) | | 3 | | 2225 | 380 | **14.1** | 14.4 | | **10.5** |

* Values are reduced with coefficients depending on the shape and height (h) of the deteriorated concrete specimens

## 31.3 NUMERICAL DETERMINATION OF THE RESIDUAL CONCRETE STRENGTH AND ASSESSMENT OF THE DEGREE OF DAMAGE

Thermal and structural response of four-bay, five-story reinforced concrete frame (only one part of the whole frame with defined support conditions) exposed to fire scenario at the two floors only, has been investigated analytically (Fig. 31.5). Elements geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio and defined fire scenario were taken into account while the nonlinear and transient temperature field and the concrete strength reduction in the cross section of the elements exposed to fire were determined.

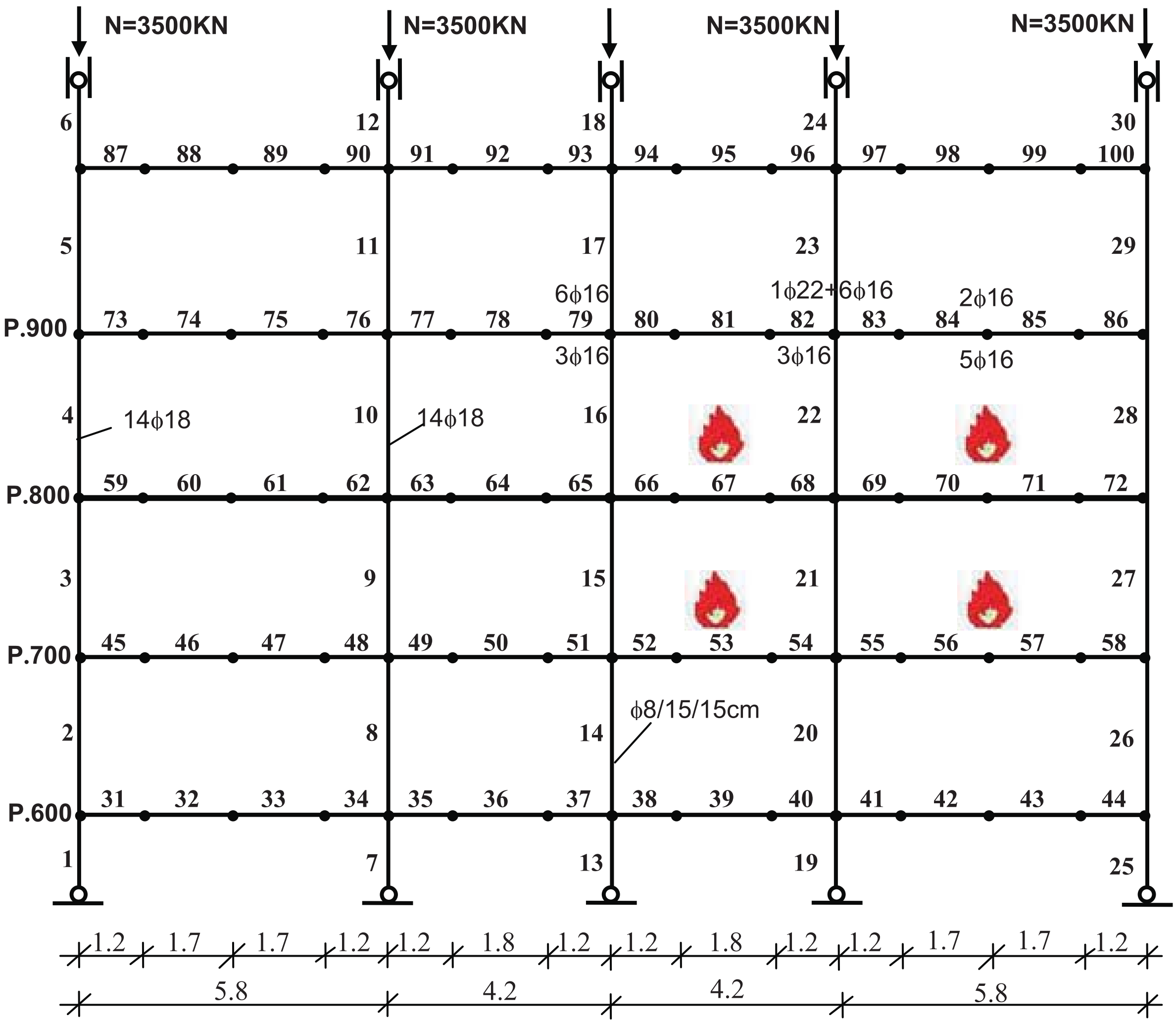


Fig. 31.5 Schematic presentation of the fired frame (elements discretization)

The computer program FIRE was used to solve this problem and the following assumptions were made:

- Fire was modeled by a single valued gas temperature history and in this case ISO 834 fire model was used. According to data gathered through detailed visual survey of the burned structural elements and the change of concrete color it was assumed that the maximum fire temperature of 1000°C was reached at the moment t=1.2hour and after that the cooling period started.
- Temperature dependent material properties were known (recommended in EC2)
- Two dimensional heat transfer was assumed.
- The fire boundary conditions were modeled in terms of both convective and radiating heat transfer mechanisms. For the surfaces directly exposed to fire the coefficient of convection was assumed $h_c$=25W/m$^2$ $^0$C and for the unexposed surfaces $h_c$=9W/m$^2$ $^0$C, as it is recommended in Eurocode 2, part 1.2.
- No contact resistance to heat transmission at the interface between the reinforcing steel and concrete occurred.
- The easy heat penetration, after cracks had appeared, or some parts of the cross section had crushed, was neglected.

Results obtained by the thermal and static analysis of this frame structure show that fire had the most negative influence upon the elements from level 800 (over the 7$^{th}$ floor). The fire intensity was less on the 8$^{th}$ floor and the degree of damage was less too.

The most damaged column is incorporated into the wall, so it was exposed to fire only from the inside of the compartment, but the temperature on the other side (in the hall) was raising proportionally to the temperature in the fire compartment (the fire flames were coming out through the open door) and the heating was from the both sides, but not with the same intensity. The dimensions of the cross section of this column (fig.6) are 60×60cm, the compressive strength of concrete before action of fire was $f_c$=40MPa. It is symmetrically reinforced with 18ϕ16. The yield strength of the reinforcing bars is $f_y$(20°C)=240Mpa. Before the action of fire the column was loaded by axial force N=3000KN and the compressive stress was 8 Mpa (20% of $f_c$).

The concrete strength was reduced as a result of high temperatures in the surface layers of the cross section (fig.6a). Calculation results indicate that on the side of fire, in 4-5cm thick layer, which is 17% of the cross section of the column, the strength reduction is significant and the residual strength of concrete is 16Mpa in average (Fig. 31.6b). These results correspond well with experimental results obtained by laboratory testing of kerns taken from the nearest RC wall (B8, Tab. 31.1). In the core of the cross section the strength of concrete is not reduced.

Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers (Fig. 31.7a and Fig. 31.7b). During the cooling phase, when the temperature in the cross section layers decreased, the negative elongation was not proportional to positive elongation when temperature increased, so it caused longitudinal cracks along the main reinforcement (Fig. 31.2c). These cracks were visually noticed on the surface of all columns, but the depth was not defined.

The reinforcing bars were close to fire and they were all the time in compression by the axial force and the action of fire. The yield strength was reduced due to the high temperatures, so the reinforcing bars started to yield very soon and high plastic deformations were noticed. During the cooling phase the stresses in reinforcement changed the sign and residual stresses in tension occurred, although they were loaded in compression (Fig. 31.8).

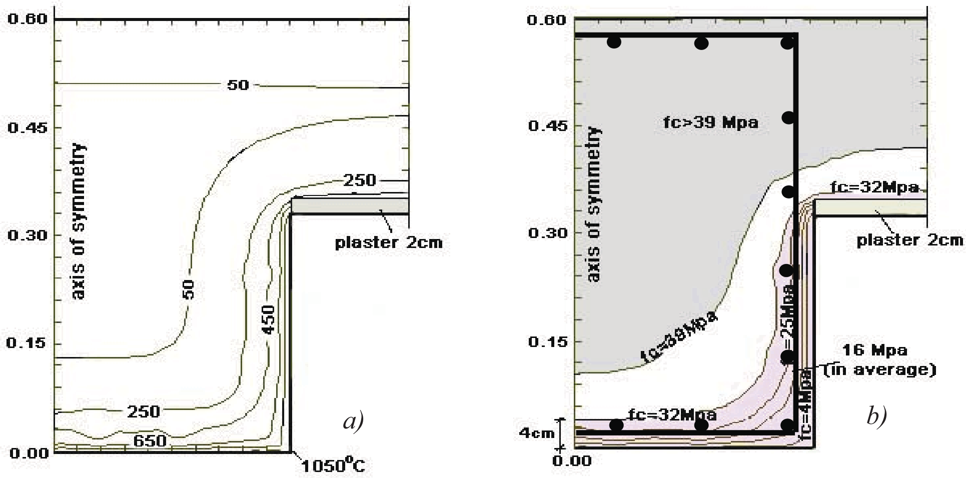


Fig. 31.6 a) Temperature distribution; b) Residual concrete strength
in the cross section of the most damaged column

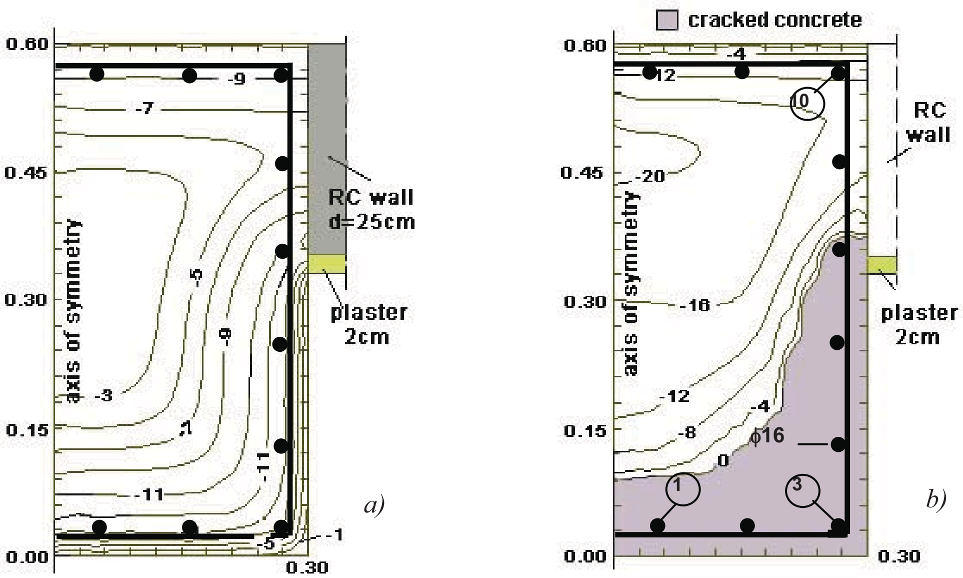


Fig. 31.7 Stress distribution in the cross section of the most damaged column
a) at max. temperatures; b) after the cooling period

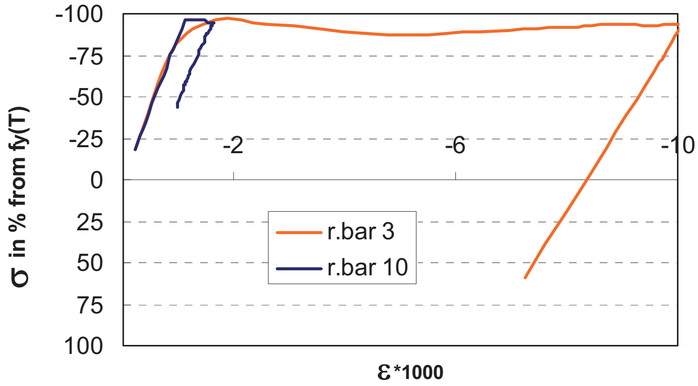


Fig. 31.8 Stresses in the reinforcing bars no.3 and no.10 of the most damaged column, as a percent of yielding strength at corresponding temperature

Columns S2 and S3 were exposed to fire almost from all sides, so it caused almost symmetrical cross section temperature field. In the cross section surface layers, as a result of high temperatures, the concrete strength was reduced. Calculation results indicate that in 4-5cm thick layer, which is 30% of the cross section of the column, the strength reduction is significant and the residual compressive strength of concrete is 15Mpa in average (Fig. 31.9b).

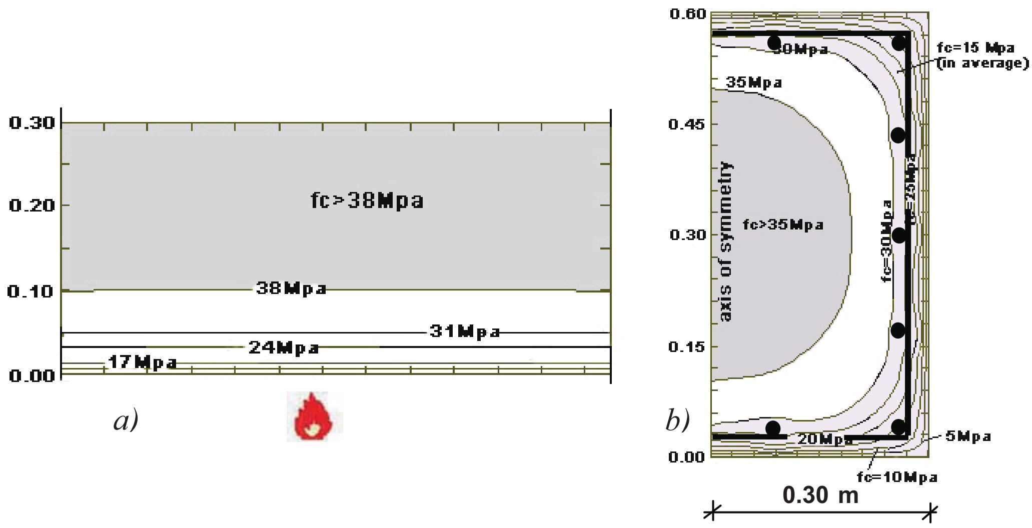


Fig. 31.9 Residual concrete strength in cross section of
a) RC wall exposed to fire from inside; b) RC column exposed to fire from all sides

Results obtained by the thermal and static analysis of the frame structure show that fire had the most negative influence upon the beam elements from level 800 (over the 7$^{th}$ floor). These results correspond well with experimental results obtained by laboratory testing of specimens taken from the elements from that floor and with visually recorded changes in concrete color (V7, G7, D7, E7, Tab. 31.1). The fire intensity on the 8$^{th}$ floor was less then that of the 7$^{th}$ floor and the degree of damage was less too (for B8 the reduction of concrete strength is less then for the other specimens). According to the defined degree of damage, beams and columns from the 7th and 8th floor have to be adequately repaired.

Beams from level 800 (over the 7th floor) were fire exposed from both sides, but from the upper side, as a part of the RC slabs, they were covered with 1cm thermal isolation and 4cm lean concrete (Fig. 31.1c), that directly influenced upon the cross section temperature field (Fig. 31.10a) and the concrete strength reduction (Fig. 31.10b). Beams from level 900 (over the 8th floor) were fire exposed only from the bottom side.

Stress redistribution was caused due to the high temperature difference between the surface layers and inner layers. During the cooling phase, when the temperature in the cross section layers decreased, the negative elongation was not proportional to positive elongation when temperature increased, so it caused longitudinal cracks along the main reinforcement. These cracks were visually noticed on the surface of all columns and beams, but the depth was not defined.

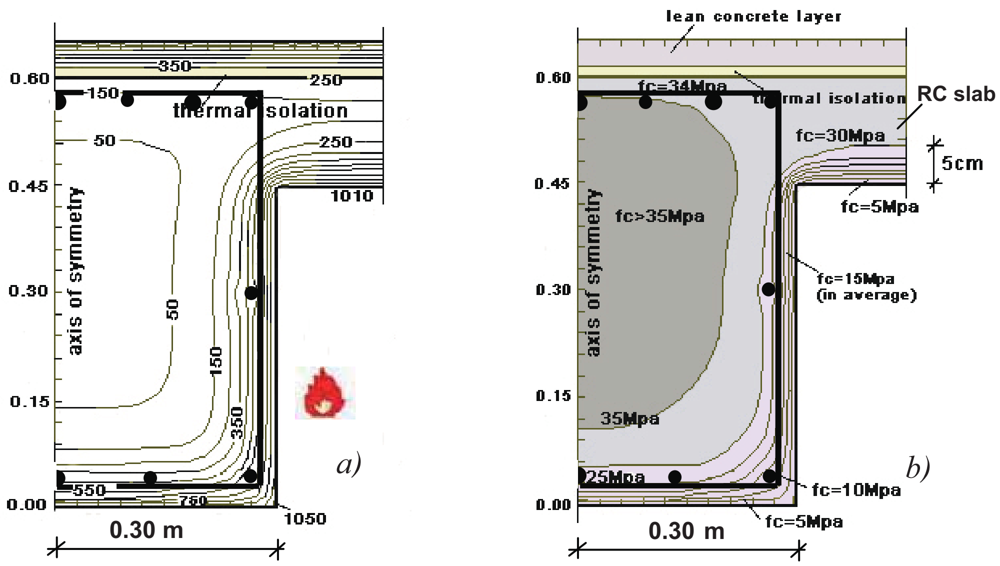


Fig. 31.10 a) Temperature distribution; b) Residual concrete strength in cross section of beam and corresponding RC slab

During the fire period large deformations occurred in beam elements from level 700 (under the fire compartment) although they were not heated because of the thermal isolation and concrete cover over the RC slabs. During the cooling period all cracks were closed and there were no additional residual stresses.

The results obtained by the thermal analysis of the RC slabs type OMNIA, the detailed visual survey in-situ and the experimental results obtained by laboratory testing of specimens (MS, SII, Tab. 31.1) lead to a conclusion that the concrete strength is reduced only in the bottom layers where concrete is in tension and has no influence on the bearing capacity of the slabs. In the upper 10cm thick compressed layers the concrete strength is not significantly reduced, therefore the recommended repair will be only surface finishing of the bottom of the slabs.

The RC walls were fire exposed only from the inside of the compartment, the temperature didn't penetrate deep in the cross sections (fig.9a) and the strength reduction was significant only in the concrete cover layers (3.0-3.5cm), therefore the repair of the RC walls is not recommended. These results correspond well with experimental results obtained by laboratory testing of specimens B8, G7, D7, E7 (Tab. 31.1) taken from the RC walls from the 7$^{th}$ and 8$^{th}$ flour. It is not the case only for the specimen V7 (Fig. 31.3a) taken from the RC wall in the room where the fire started and the temperature was highest.

### 31.4 SUGGESTED REPAIR OF FIRE DAMAGED STRUCTURAL ELEMENTS

The aim of the repair suggested in this project is to obtain nearly the same strength, stiffness, deformability and ductility of the repaired elements and their cross sections as they had before the fire action. The bearing capacity of the column cross section can be determine from the interaction diagrams: moment-axial force (M–N), moment-curvature (M–φ) and axial force-curvature (N–φ). The main interaction diagram for the beam cross section is moment-curvature (M–φ).

The M-N interaction diagrams of the most fire damaged column before and during the fire action, after the cooling phase and after the suggested repair are presented on Fig. 31.11, Fig. 31.12 and Fig. 31.13.

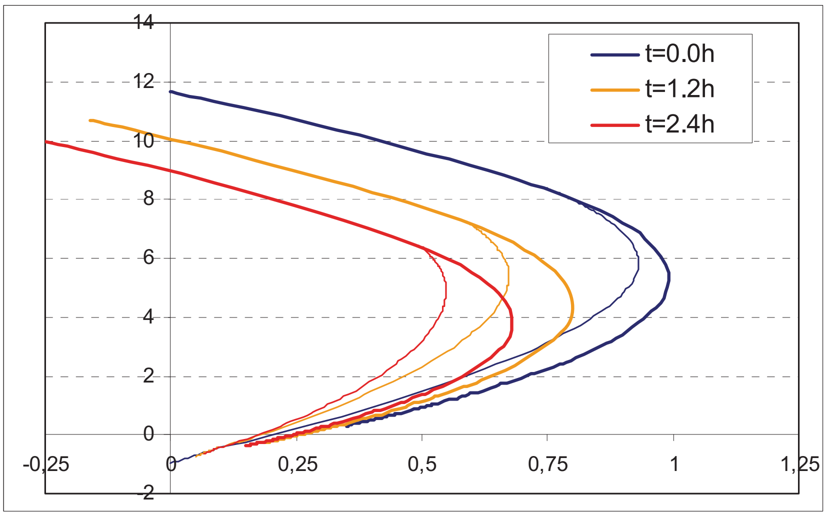


Fig. 31.11 Interaction diagram “bending moment-axial force” for column S4

(before action of fire and at the moment of max. temperature)

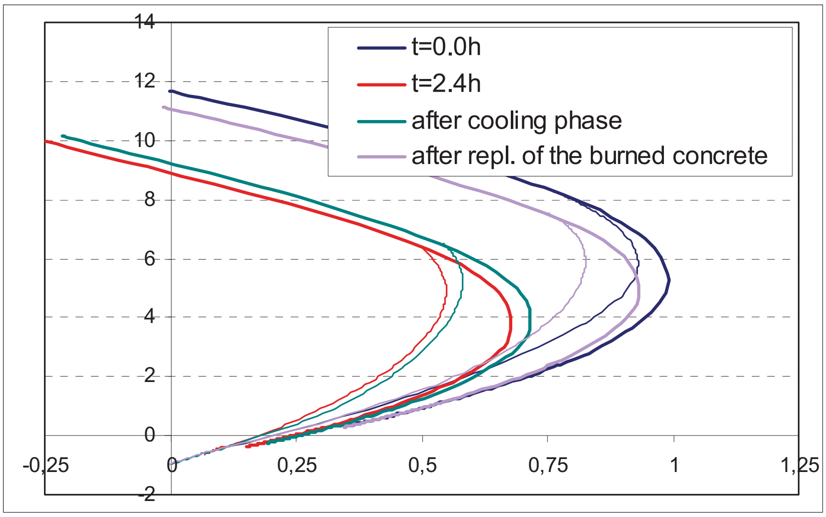


Fig. 31.12 Interaction diagram "bending moment-axial force" for column S4 (before action of fire, at the moment of max. temperature, after cooling phase and after replacement of 5cm thick layer of burned concrete)

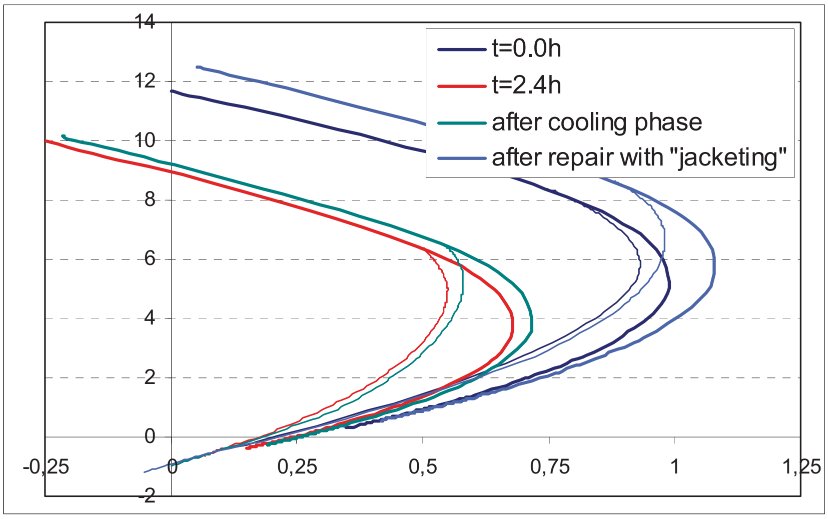


Fig. 31.13 Interaction diagram "bending moment-axial force" for column S4 (before action of fire, at the moment of max. temperature, after cooling phase and after suggested repair with "jacketing")

The interaction diagrams indicate negligible increase in bearing capacity and ductility of the cross sections after the cooling phase. Replacement of the deteriorated 4-5cm thick concrete layers with new concrete layers has a negligible effect too, therefore the suggested repair is: replacement of the deteriorated 4-5cm thick concrete layers with new concrete layers; addition of main reinforcement and stirrups, addition of new lateral 3cm thick concrete layers. This method is well known as "**jacketing**" and it is most effective if the new layers are made from all sides (the jacket is closed). In this case, due to the elements position, jacketing is not possible from all sides and the suggested solution is jacketing of columns only from three sides, or partly from the fourth side but the jackets will not be completely closed, and for beams jacketing is only from the three sides.

The suggested repair of the most damaged column is presented in Fig. 31.14. The repair consists of: elimination of the "burned" 3-4cm thick concrete layers; addition of main reinforcement (4ϕ18, RA 400/500-2) in the corners of the cross section and at the contact with the RC wall; addition of stirrups (ϕ10/7.5/15cm, GA 240/360) and addition of new lateral 3cm thick concrete layers. Due to the specific geometry open stirrups are used and they have to be welded to the main reinforcement in the RC wall (according to the situation in-situ). Welding between the new ribbed reinforcement and existing smooth reinforcement is not recommended for two reasons: the additional reinforcement has higher yield strength and the existing smooth reinforcement has residual plastic deformations.

The repair of the beam elements consists of: elimination of the "burned" 3-4 cm thick concrete layers; addition of reinforcement in the two corners of the cross section (2ϕ20, RA 400/500-2); addition of stirrups and new lateral 3cm thick concrete layers (Fig. 31.15). Due to the specific geometry open stirrups are used and they have to be welded to the existing stirrups at the middle of the cross section.

The jacketing has to be made with **self compacting** concrete MB35, or **torcrete** concrete MB30.

The recommended repair for the RC slabs will be only surface finishing of the bottom of the slabs, and repair of the RC walls is not necessary.

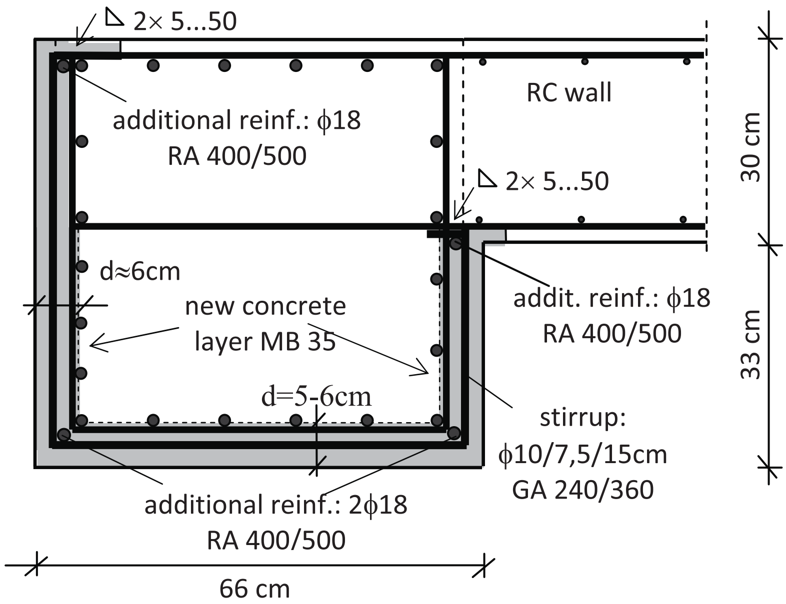


Fig. 31.14 Repair of RC column by "jacketing"

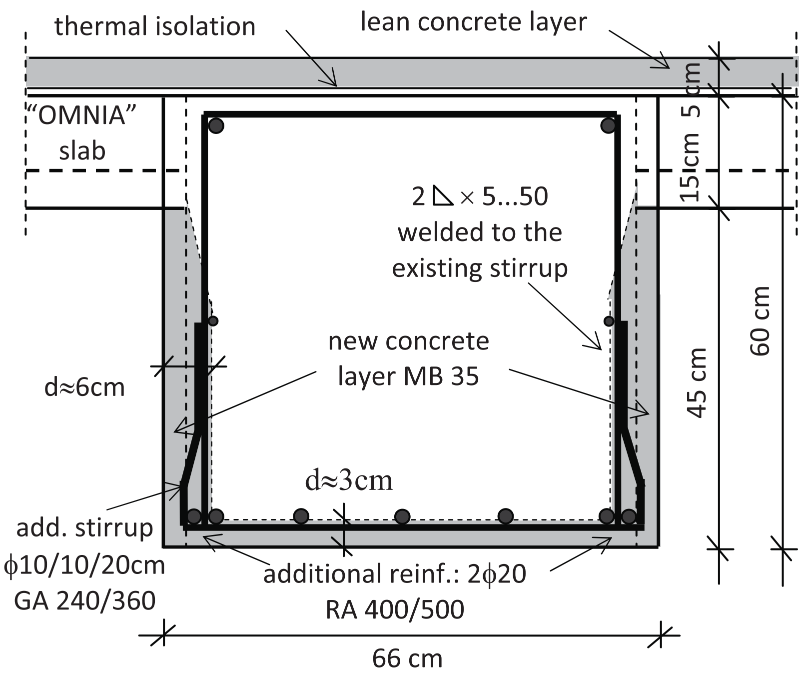


Fig. 31.15 Repair of RC beam by "jacketing"

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# 32 SAFETY ASSESSMENT OF STEEL STRUCTURE OF THE SINGLE-STOREY INDUSTRIAL BUILDING AFTER A LOCAL FIRE

Summary

Single-storey, two-bay industrial steel-framed building with columns rigidly mounted in the foundation and roof trusses pivotally supported on the columns was subjected to a local fire. Some polypropylene tanks stored in piles near the extreme pillars of the left side of the building has completely burned. Unfavourable events occurring during the action of heat on these above mentioned two steel columns have had a direct impact on the entire static system, and thus – on the whole structural elements located not only in the right bay (crane beams, cold-rolled purlins, roof and wall bracing systems, skylights, etc.) but the rest of the building, as well. Steel construction of the hall was not protected against the fire temperatures at all.

Main structural components of the building have been subjected to high temperatures of varying degrees, depending on their location in the structural system. None detailed data are available regarding either the time-length of the fire itself or the distribution of the temperature field. Local Fire Department has not provided this sort of information. Further increase of internal temperature was observed during the flashover phase after the unsealing of finishing layers of the roof covering and after the bursting of the oblong side windows in the roof skylights.

The approximate temperature of the fire environment was estimated based on the analysis of available source materials and studies on this topic. During the study (following the information provided by available literature) it was adopted that the combustion temperature of polypropylene (PP) can reach 673.80$^{o}$C. This means that the significant part of steel structural components located in the zone of high temperatures during the fire have had greatly reduced strength values. Basing on the impact analysis of high temperatures a required range of modernization for steel framework has been specified.

## 32.1 GENERAL INFORMATION

### 32.1.1 General description of the building

The property described in this paper is a single-storey, two-bay industrial hall equipped with two gantry cranes and skylights in the roof. The height of the building measured to the ridge of trusses is equal approximately +9.945 m, and to the ridge of skylights reaches +12.035 m. The spam of naves, measured in axes of columns is equal L= 2 x 18,0 m. Totally, the main steel load-bearing structure of the building consists of 17 two-bay steel frames.

The construction of the roof was design in form of flat roof trusses made of thin-walled elements, which were produced of unalloyed carbon steel, grade St38U-2 due to DIN standards. The roof trusses were technologically divided into halves and assembled during erection. The bolted assembly connections localised in a mid-spam of upper and lower chords have been designed as pre-stressed with high-strength bolts.

Steel columns, designed as two-chord laced built-up members with the cross-section dimensions dependent on the location in the structure, have been made of thin-walled elements produced of unalloyed carbon steel, grade St38U-2 due to DIN standards. These columns have been composed of two parts of different cross-sectional dimensions: the upper one that supports the roof trusses, and the lower part which supports the crane beams. In a roof slope some transverse bracing systems, distributed at every 6 fields, have been mounted. No longitudinal roof bracing systems have been provided. Bracing members were made of thin-walled angles manufactured of steel grade St38U-2. The roof bracings of the same sort are provided for skylights. Roof purlins were designed as continuous beams made of thin-walled members.

The project of the building has not contained any calculations or guidelines for fire protection of steel structure.

### 32.1.2 Description of structure after fire

The fire took place on 1st of August 2007. In a result, few lacquer tanks made of polypropylene which were located near the outer longitudinal wall, were completely burnt, (Fig. 32.1.1, 32.1.2).

Fig. 32.1.1


Fig. 32.1.2

Unfortunately, no data are available, how long the fire lasted, and what temperature prevailed in the hall during a fire. As it was mentioned before, the steel structure of the object was not protected against high temperatures at all. The upper parts of two neighbouring columns, located near the fire source were strongly damaged. Some structural components of columns such as diagonals and posts have been locally deformed and destroyed, (Fig. 32.1.3).


Fig. 32.1.3

Under the influence of high temperature also some significant deformations of components in two roof trusses, localised nearest to the fire source occurred, (Fig. 32.1.4). Partial deformation of some diagonals and fragments of upper chords was also found in other roof trusses, more distant from the source of fire.

Fig. 32.1.4

## 32.2 ANALYSIS OF THE STRUCTURE SUBJECTED TO FIRE

### 32.2.1 Evaluation criteria

Fire Service Department has not defined the range of temperatures that prevailed in the hall during a fire - there are completely no data on this subject. Reliable estimation of the actual temperature of fire environment is very difficult and could be done only with the help of advanced numerical tools. Based on the analysis of source materials and available studies on the subject (references) it may be determined that the combustion temperature of combustion of polypropylene (PP) can reach 673.80°C, (Tab. 32.2.1).

Tab. 32.2.1 Maximum temperature of the hot zones in a fire compartment

| POMIESZCZENIE | Maks. temp. strefy gor acej [°C] | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Dd | PA | PE | PMMA | PP | PU | PS | PWC |
| pokój z pożarem | 609,3 | 662,5 | 682,1 | 554,6 | 673,8 | 667,8 | 839,0 | 423,2 |

### 32.2.2 Strength analysis

Thermal interactions result in significant changes in mechanical properties and strength of structural steel. The reduction factors for the stress-strain relationship for steel at elevated temperatures, quoted after Eurocode 3: EN 1993-1-2 are given in Tab. 32.2.2. Linear interpolation may be used for values of the steel temperature intermediate to those given in table.

For the temperature level reaching a value of approximately 673.8°C the change of these parameters, in relation to ambient temperature, averages:

- approx. 18% (elastic modulus at elevated temperature $E_{a,\Theta}$, relative to $E_a$),
- approx. 10% (proportional limit at elevated temperature $f_{p,\Theta}$, relative to $f_y$),
- approx. 30% (effective yield strength at elevated temperature $f_{y,\Theta}$, relative to $f_y$).

Tab. 32.2.2 Reduction factors for stress-strain relationship of carbon steel at elevated temperatures

| Steel temperature $\theta_a$ | Reduction factor (relative to $f_y$) for effective yield strength $k_{y,\theta} = f_{y,\theta}/f_y$ | Reduction factor (relative to $f_y$) for proportional limit $k_{p,\theta} = f_{p,\theta}/f_y$ | Reduction factor (relative to $E_a$) for the slope of the linear elastic range $k_{E,\theta} = E_{a,\theta}/E_a$ |
|---|---|---|---|
| 500 °C | 0,780 | 0,360 | 0,600 |
| 600 °C | 0,470 | 0,180 | 0,310 |
| 700 °C | 0,230 | 0,075 | 0,130 |

This leads to the conclusion that a significant part of steel structure components located in the zone of high temperatures, as a result of the fire has greatly reduced or completely lost its capacity.

The result of the significant reduction of the load-bearing capacity of structure is well seen in form of local deformations and damages of members and components.

In the absence of detailed data on the temperatures that prevailed inside the object during the fire during the analysis it was assumed that:

- the highest temperatures prevailed in the zone located just below the ceiling,
- the temperature near the source of fire ranged within the limits 400-600 °C,
- all the bolted assembly connections localised in a mid-spam of upper and lower chords, designed as pre-stressed with high-strength bolts, have been subjected to destructive influence of high temperatures.

### 32.2.3 Residual strength of steel structures after fire

Although the ad hoc capacity of steel structures decreases due to strength loss of structural steel when the temperature rises the residual strength of steel seems to recover quite well after cooling down, as it was proved in tests (Kirby at al., 1986 & Outinen, 2007). The residual strength of steel after cooling down depends on many parameters, such as: original properties of the steel grade, the maximum reached temperature, loading history, degree of deformation, etc. The general conclusion from the limited amount of experimental results concerning the residual strength of structural steel after heating leads to the statement, that if the elements of steel structure are not deformed in fire, the strength of steel will likely be still adequate, but all the connections, surface coating etc. Have to be checked thoroughly. A rough limit is drawn to about 600°C after which permanent loss of strength seems to take place. According to BS 5950-8:2003 hot finished steels and cast steels can be re-used after fire when the deformations remain within the acceptable tolerances for straightness and shape. For cold finished steel grades that remain within tolerance, it's recommended to assume they have approx. 90% of the original strength.

**32.3 CONCLUSIONS**

1. The design project of the presented building haven't included any calculations, or guidance on fire protection, and steel structure was not protected against fire at all.
2. Fire Service has not specified the temperature field distribution in fire-stricken areas. In Polish realities they are not used to do it, in general.
3. Based on Eurocode 3 one can determine in an approximate way some changes of the significant mechanical parameters and strength of steel at high temperature, which allow the estimation of the real load-bearing capacity and safety of structures as well. This possibility particularly applies to the structural elements more distant from the source of fire.
4. Structural elements located close the source of the fire, and in a lesser extent, the remaining steel structure have lost their load-bearing capacity.
5. The pre-stressing force in high-strength bolts used in assembly connections of roof trusses has been degraded under the influence of high temperature.

Taking the items given above into consideration:

- In case of new buildings (including also existing ones, for which the change of function is possible during their lifetime) it should be obligatory to determine how the high temperature may influence the structure, and execute the project design taking into account the real fire-load density and specifying the necessary fire protection,
- State Fire Service should at least in approximate range determine the temperature field distribution in fire-stricken premises.

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# 33 THERMAL ANALYSIS OF A DOUBLE DECK BRIDGE

Summary

The goal of the present case study is to present the thermal study of different components of a double deck bridge, for establishing the temperature evolution in a fire situation. Unprotected as well as protected cases are analysed using a protection material described in detail in the paragraph entitled “The Materials”. Three structural elements have been submitted to a natural fire curve obtained through a CFD analysis given in detail in the paragraph entitled “The Fire”.

## 33.1 INTRODUCTION

The goal of the present project is the thermal study of different components of a double deck bridge, for establishing the temperature evolution in a fire situation. Unprotected as well as protected cases are analyzed using a protection material described in detail in the paragraph entitled “The Materials”.

Three structural elements have been submitted to a natural fire curve obtained through a CFD analysis given in detail in the paragraph entitled “The Fire”.

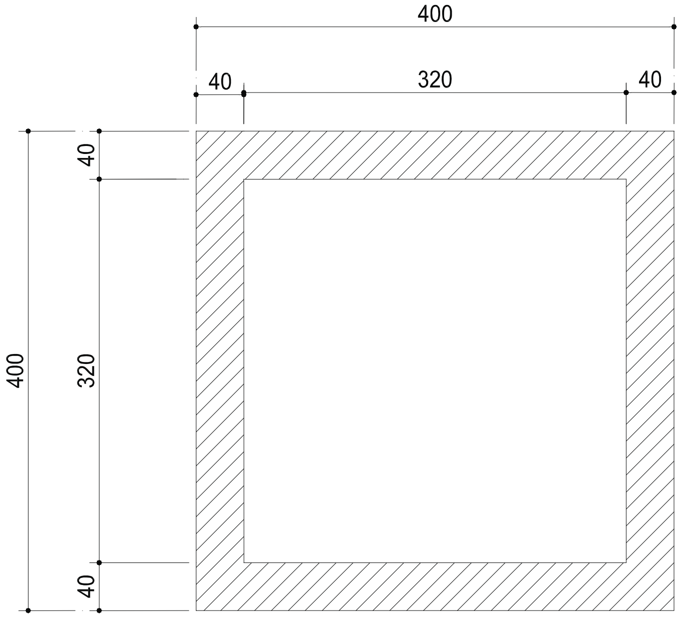


Fig. 33.1 The diagonal of the lattice girder

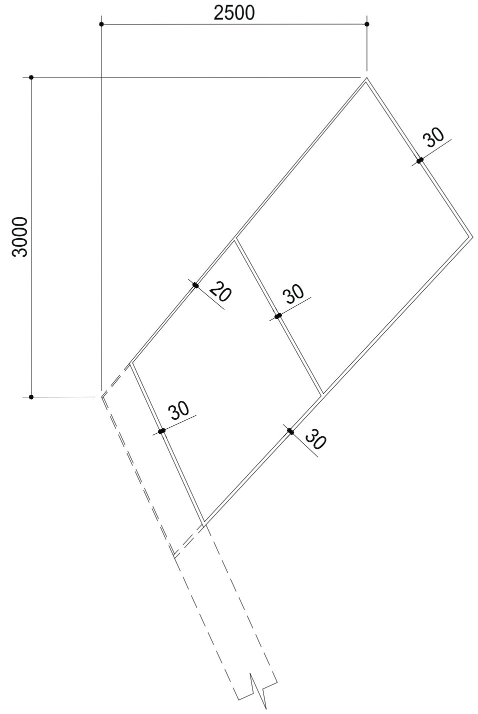


Fig. 33.2 The box girder

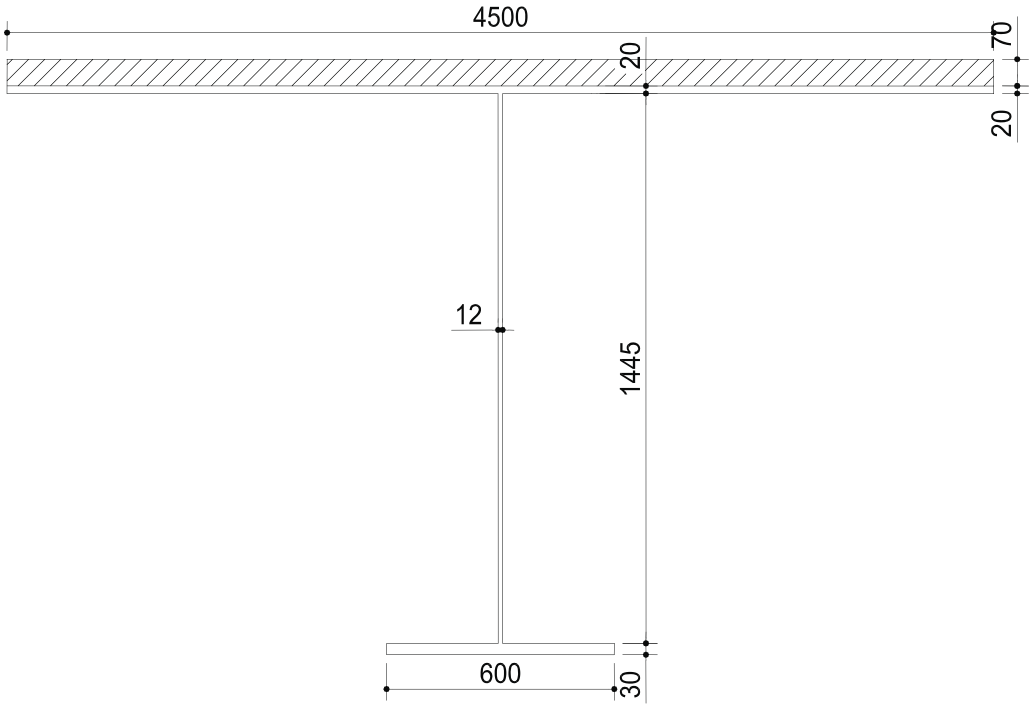


Fig. 33.3 The upper crossbeam

- The first structural element is a diagonal from the lattice girder (see Fig. 33.1).
- The second structural element is the box girder (see Fig. 33.2)
- The third element is the upper crossbeam (see Fig. 33.3)

Since the crossbeam has different widths of the upper flange and web on its span, a conservative beam is analysed having the minimum flange and web thicknesses (see Fig. 33.3).

## 33.2 THE MATERIALS

The material laws and physical principles of the thermal analysis are based on the Eurocode ENV 1993-1-2 and 1994-1-2.

Therefore the thermal properties of the steel for the diagonal of the lattice girder, the box girder and the upper crossbeam are those given in Eurocode ENV 1993-1-2.

Convection coefficients for the hot surfaces of 50 $W/m^2K$ and for the cold surfaces of 9 $W/m^2K$ are used. The relative emissivity is 0.5 for steel.

On the upper crossbeam the sheathing is assimilated to a siliceous concrete from Eurocode 1994-1-2, with a water content of 26 $l/m^3$, similar convection coefficients on the cold and hot surfaces as for the steel and a relative emissivity of 0.56.

The protection material that is used for the protected simulations, PROMATECT has the following thermal properties according to "Promat HTI": a constant specific heat of 1130 J/kg K, a constant density of 700 $kg/m^3$. No water is considered in this material, the convection coefficients for the hot and the cold surfaces are those from the steel and concrete elements. The relative emissivity is 0.56 as for the concrete material. The single property that has a linear distribution is the thermal conductivity. The manufacturer of the PROMATECT gives the thermal conductivities only up to 600°C. For this study we have extrapolated the thermal conductivity values up to 1200°C (see Tab. 33.1).

Tab. 33.1 The thermal properties of the protection material (PROMATECT)

| Temperature [°C] | Thermal Conductivity [W/m K] | Specific Heat [J/kg K] | Density [$kg/m^3$] |
|---|---|---|---|
| 0 | 0.10 | 1130 | 700 |
| 200 | 0.12 | 1130 | 700 |
| 400 | 0.14 | 1130 | 700 |
| 600 | 0.16 | 1130 | 700 |
| 800 | 0.18 | 1130 | 700 |
| 1000 | 0.20 | 1130 | 700 |
| 1200 | 0.22 | 1130 | 700 |

## 33.3 THE FIRE

Up to three fire curves have been applied on the cross sections of the structural elements. The base curve obtained through a CFD computation is the curve entitled F1100 (see Fig. 33.4) which reaches a maximum temperature of 1142°C. The other two curves are obtained from the base curve by interpolating the values as to reach a peak value of 800°C (the curve F800) and 400°C (the curve F400). After 40 minutes all three curves remain constant at 20°C.

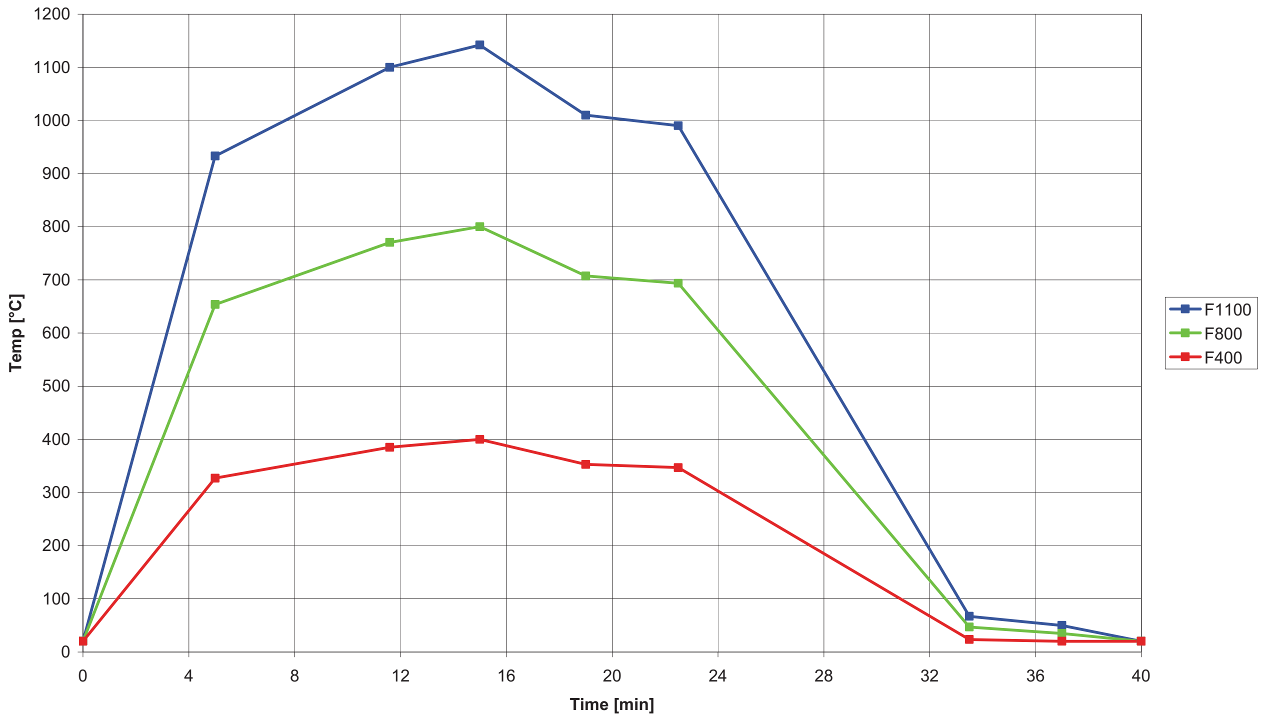


Fig. 33.4 The fire curves used in the analysis

The following three figures show the boundary conditions considered on each of the cross sections (thermal boundary conditions).

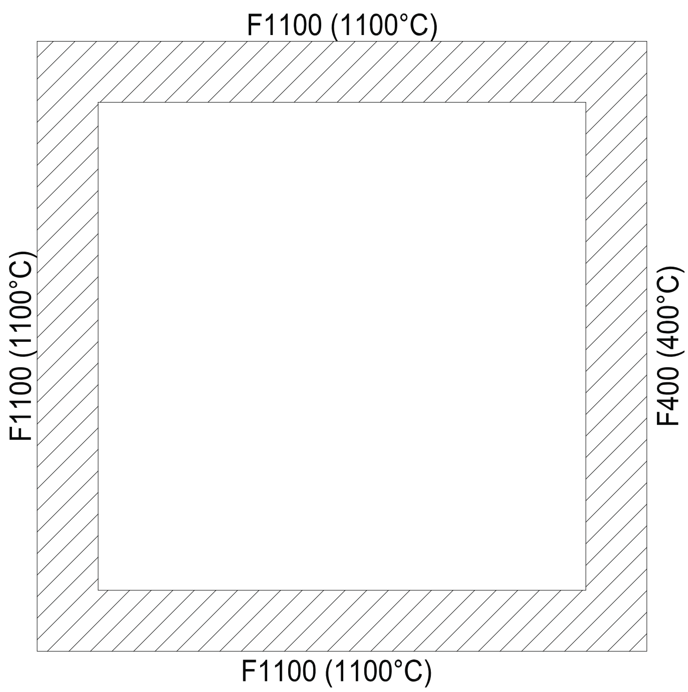


Fig. 33.5 Applied fire curves on the protected and unprotected diagonal

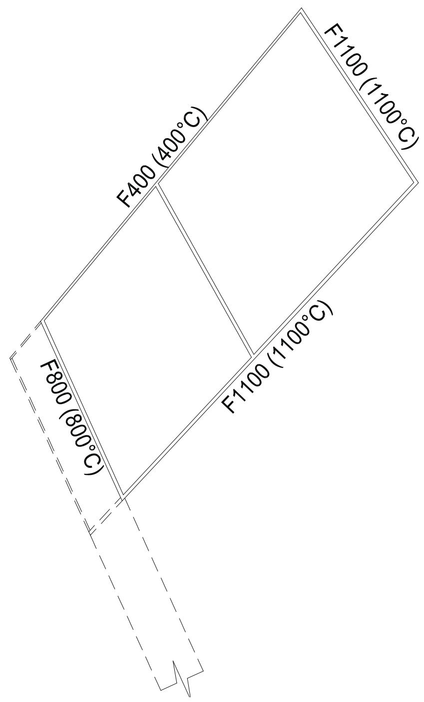


Fig. 33.6 Applied fire curves on the protected and unprotected box girder

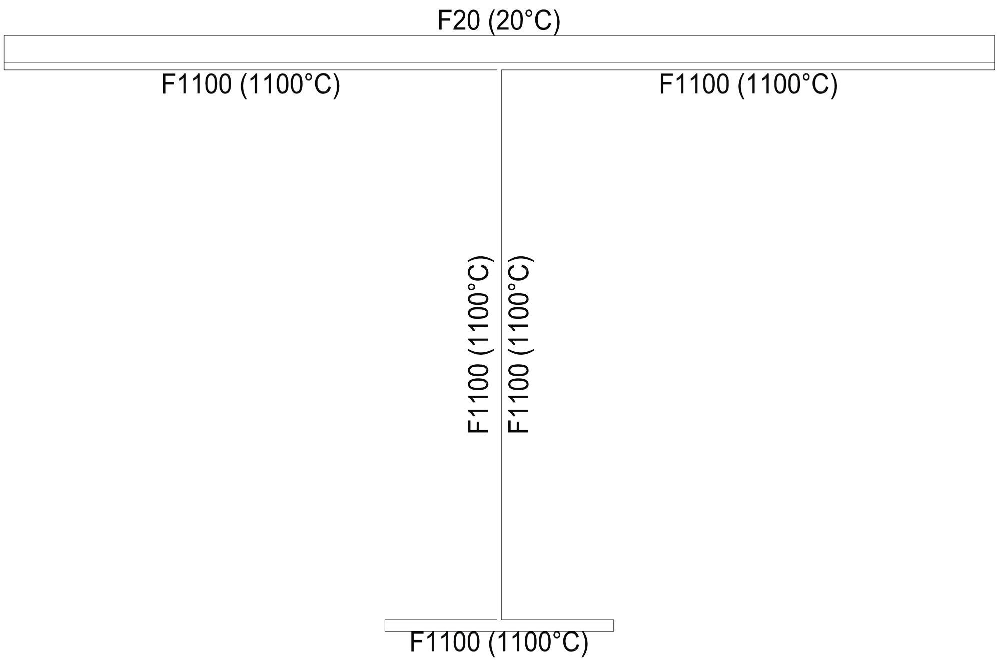


Fig.33.7 Applied fire curves on the unprotected crossbeam

### 33.4 MESHING AND PROTECTION

All the models are meshed using SAFIR 2D SOLID elements with four nodes. A rather regular, Cartesian mesh is sought for easier processing of the results of the analysis. A sufficiently fine mesh for each case is used for capturing with good accuracy the temperature distribution having a good ratio computation time/mesh.

All the figures that follow show the mesh of the cross section, the materials used, and the frontiers (applied fire conditions on the cross section).

The diagonal of the lattice girder is analysed in two cases, an unprotected case, with the fire applied on the bare steel of the hollow tube (see Fig. 33.8), and a protected case with a 15 mm box protection using the PROMATECT material presented above (see Fig. 33.9).

The box girder is analysed in two cases, an unprotected case with the fire applied on the bare steel, on the contour of the box girder and a box protected case using a 15 mm PROMATECT on the contour.

The crossbeam is analysed in four configurations: Case A – an unprotected steel beam with the fire (F1100) engulfing the lower part of the girder (see Fig. 33.10), Case B – a ceiling protection using a 15 mm PROMATECT protection, with the fire (F1100) applied on the lower edge of the protection (see Fig. 33.11), Case C – a box protection around the girder with 15 mm of PROMATECT (see Fig. 33.12), and Case D – a box protection around the girder extending on the lower edge of the upper flange (see Fig. 33.13).

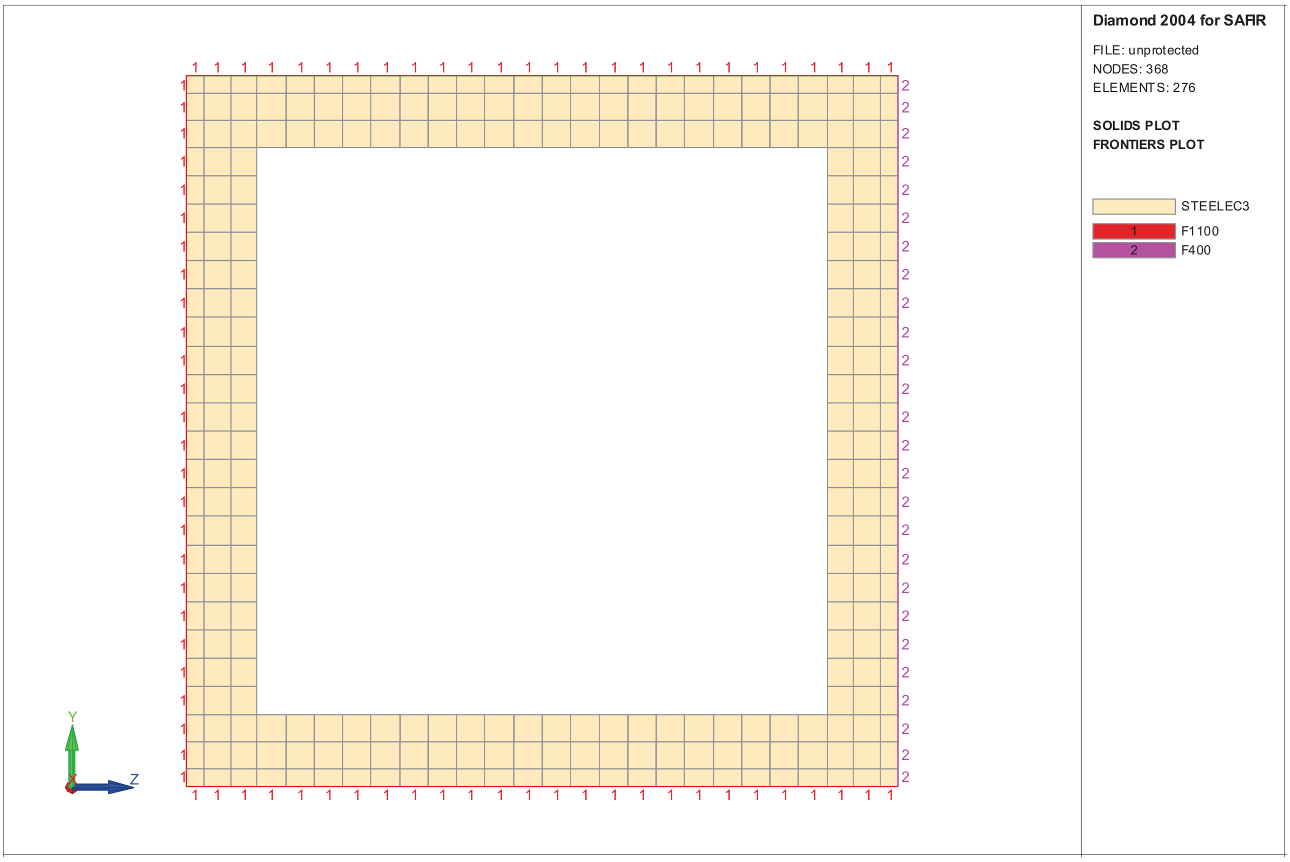


Fig. 33.8 Mesh, materials and frontiers on the unprotected diagonal of the lattice girder

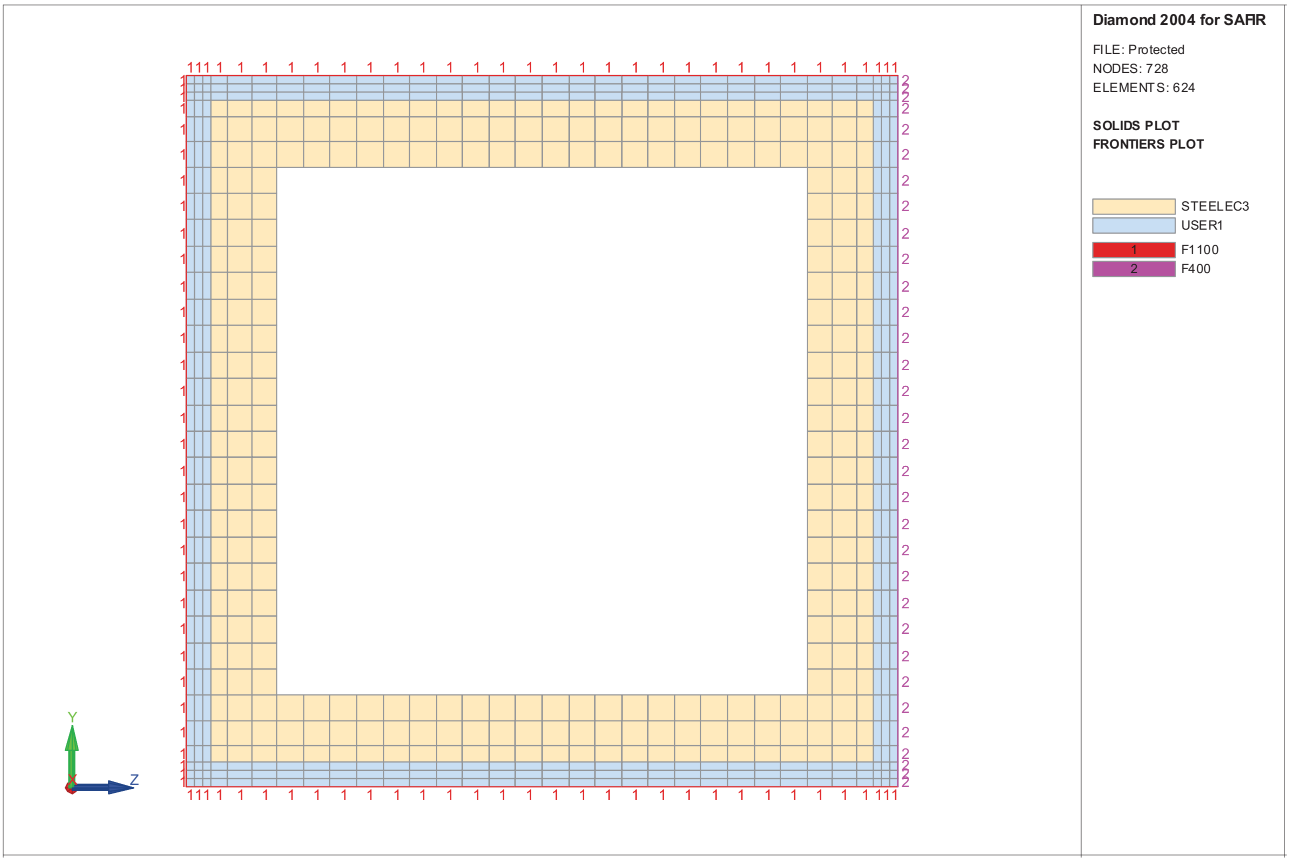


Fig. 33.9 Mesh, materials and frontiers on the protected diagonal of the lattice girder

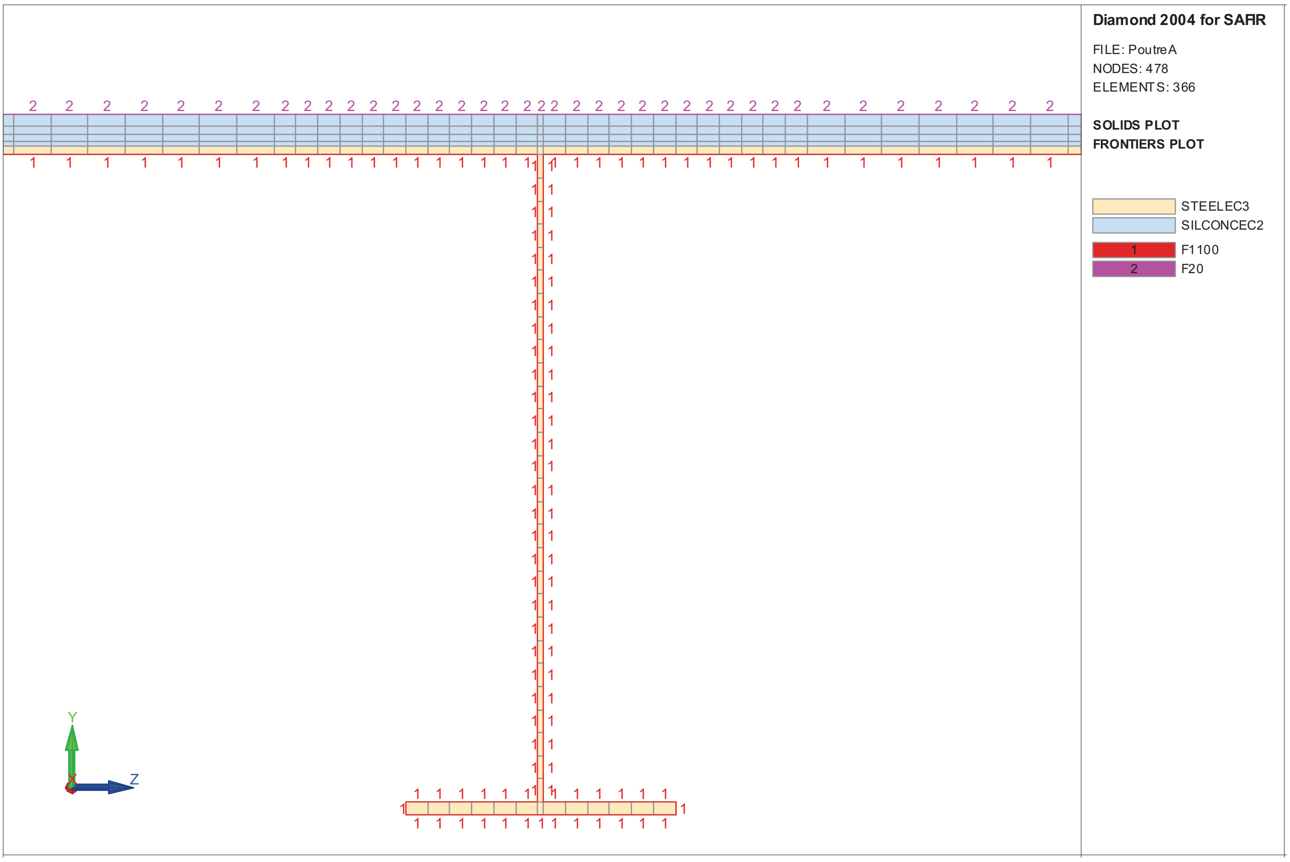


Fig. 33.10 Mesh, materials and frontiers on the unprotected crossbeam (case A)

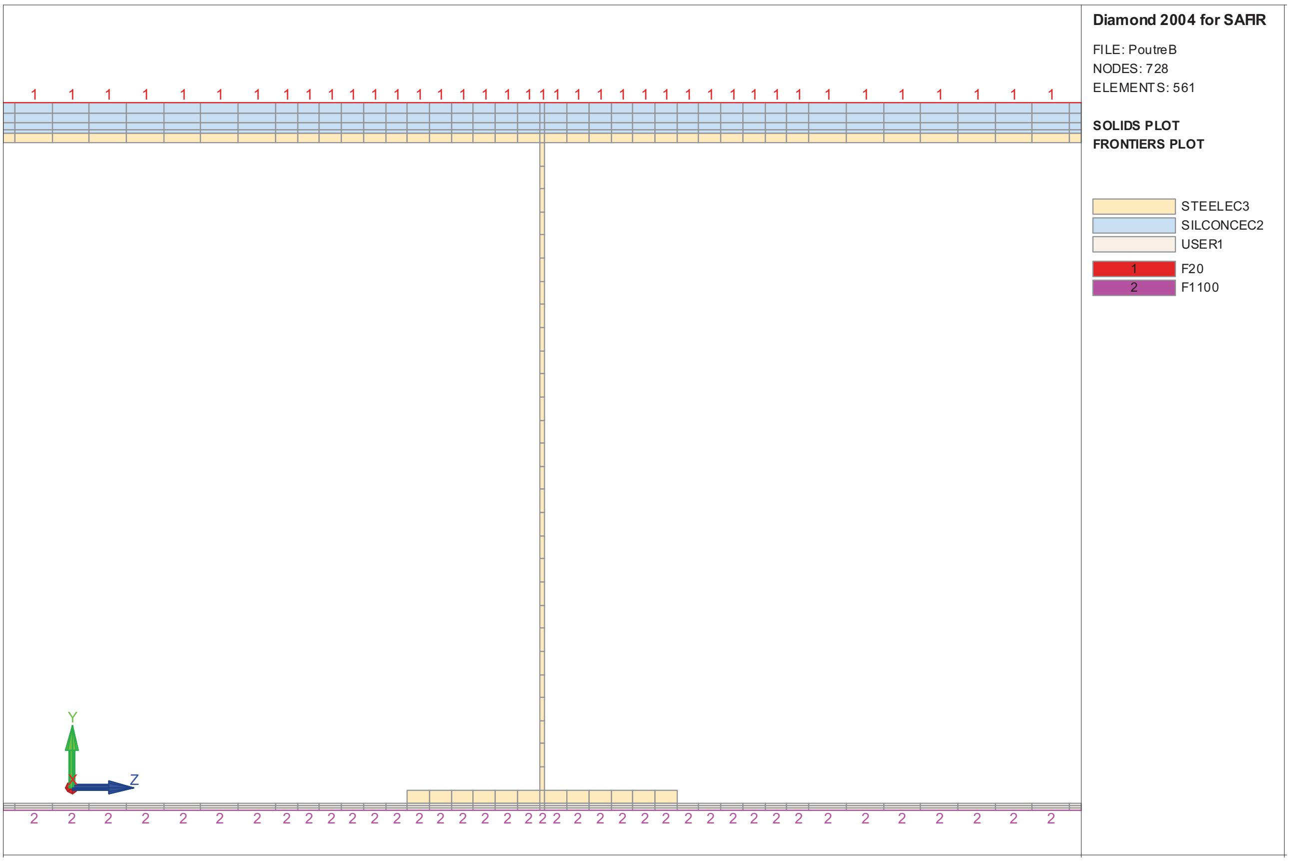


Fig. 33.11 Mesh, materials and frontiers on the ceiling protected crossbeam (case B)

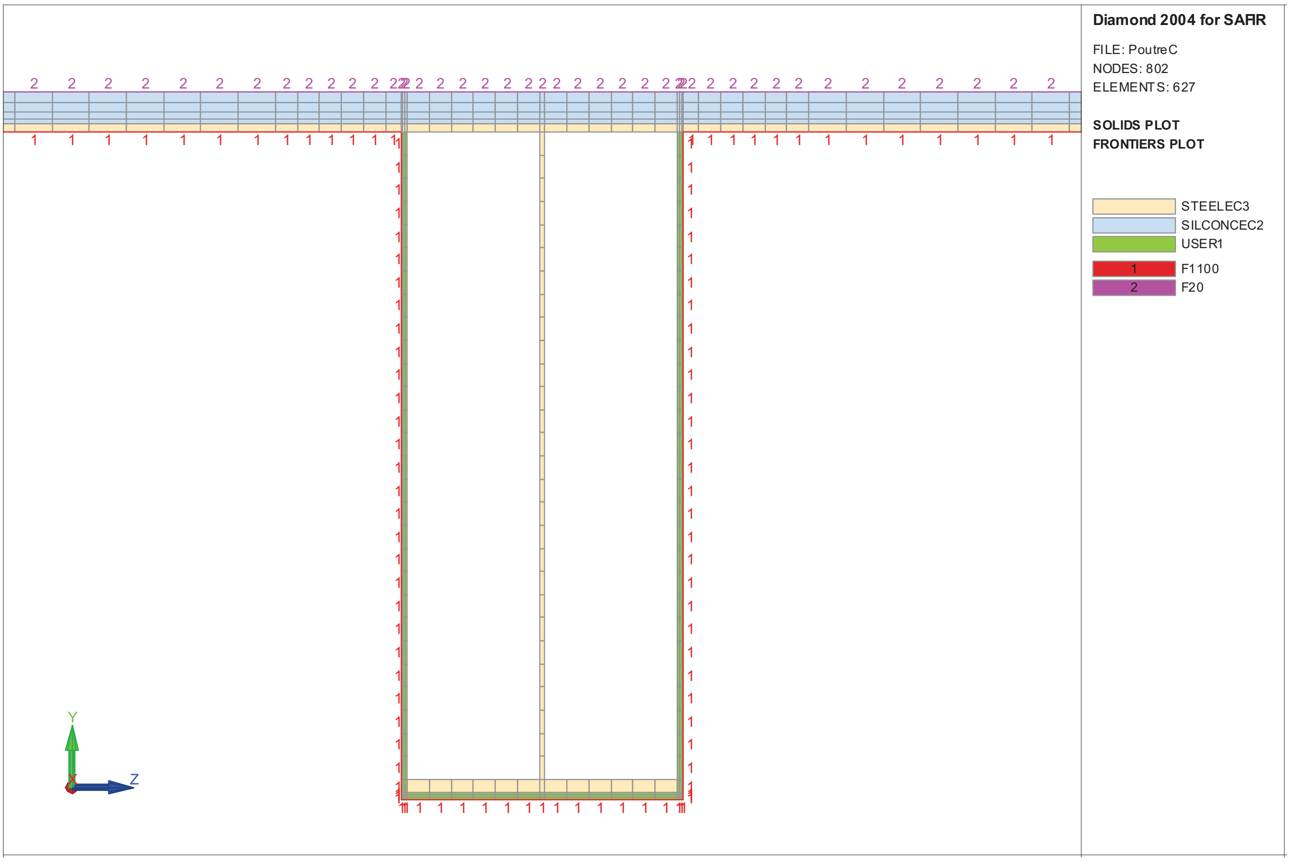


Fig. 33.12 Mesh, materials and frontiers on the box protected crossbeam (case C)

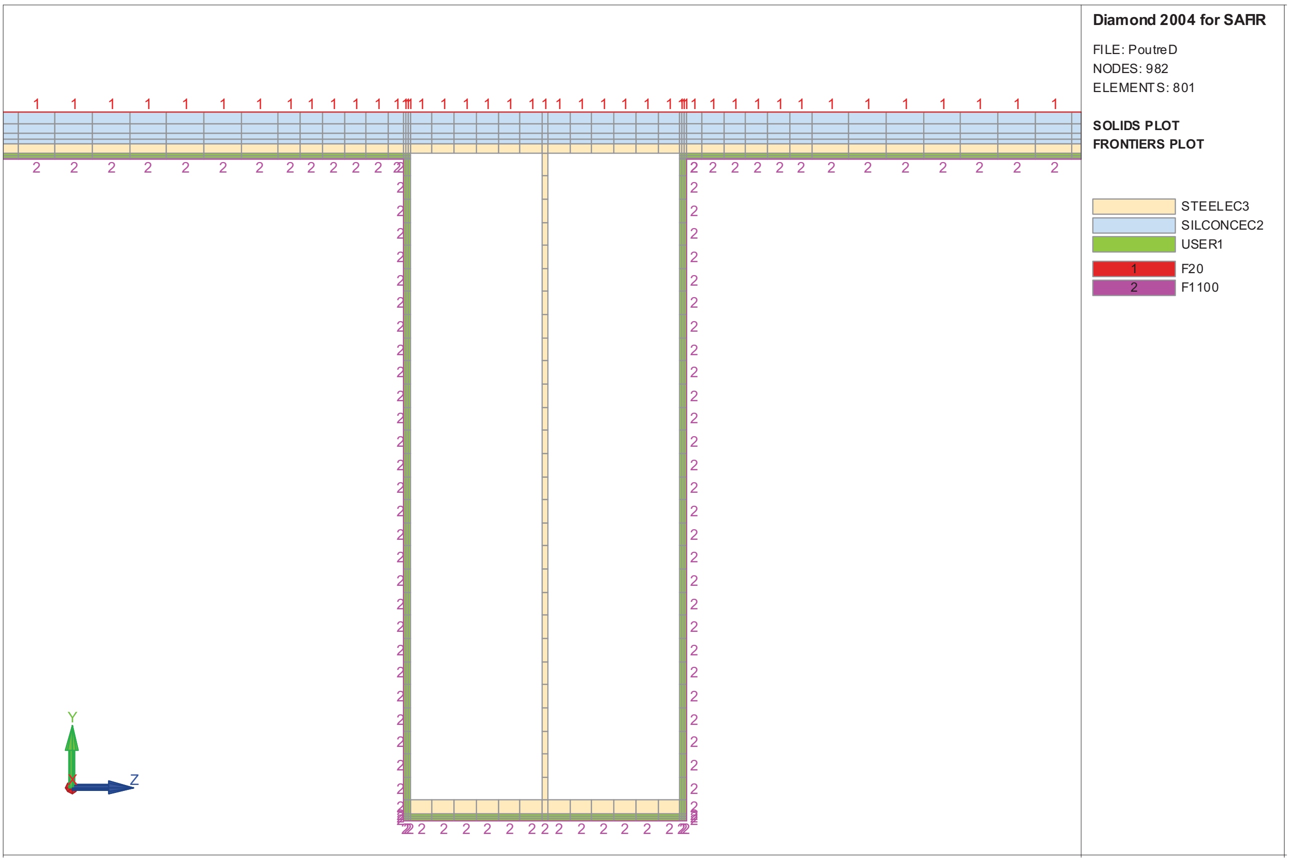


Fig. 33.13 Mesh, materials and frontiers on the box protected crossbeam (case D)

## 33.5 RESULTS

Since the fire curves applied on the cross sections have all a descending branch, the cross sections reach a maximum temperature at a certain time. The temperatures decrease afterwards more steeply for the unprotected sections and slowly for the protected ones.

The unprotected diagonal of the lattice girder reaches its maximum temperature of 780°C at 22.5 minutes (1350 sec). The temperature evolutions in four representative points on the cross section (see Fig. 33.14) are shown in the Fig. 33.15.

On the protected cross section the maximum temperature of 125.4°C is reached after 32.5 minutes (2000 sec), (there is a delay in the heating of the cross section due to the protection). While the exposed side to the lower temperatures (F400) tends asymptotically towards 90°C after 1 hour of fire exposure, the rest of the cross section is already in the cooling phase (see Fig. 33.16).

The unprotected box girder reaches its maximum temperature of 798°C at 22.5 minutes (1350 sec), a steep descending phase following this peak. For the protected box girder, with a 15 mm PROMATECT protection, the temperatures do not exceed 145°C, and after a heating phase of 33.5 minutes (2000 sec) when the maximum temperature of 143.8°C is reached, the cross section starts to cool slowly. No figures are shown.

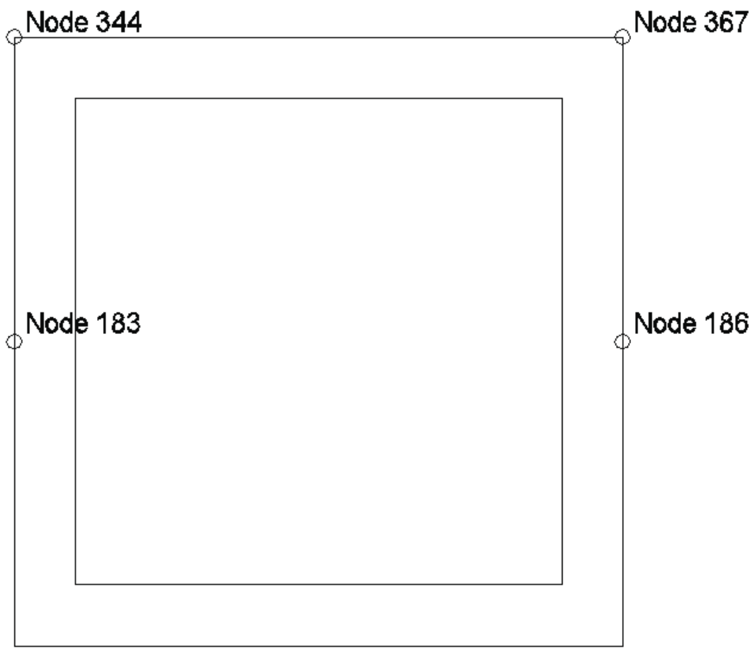


Fig. 33.14 The representative points on the cross section where temperature evolution in sought

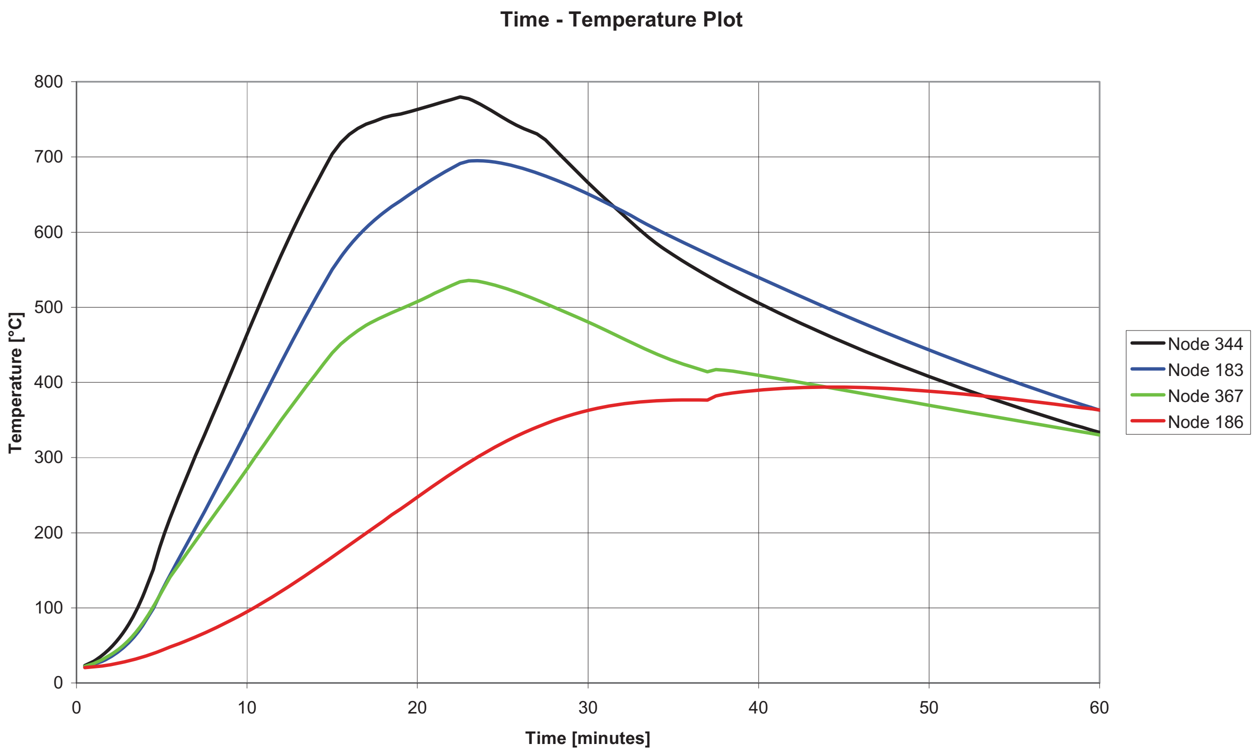


Fig. 33.15 Temperature evolution on the unprotected diagonal of the lattice girder

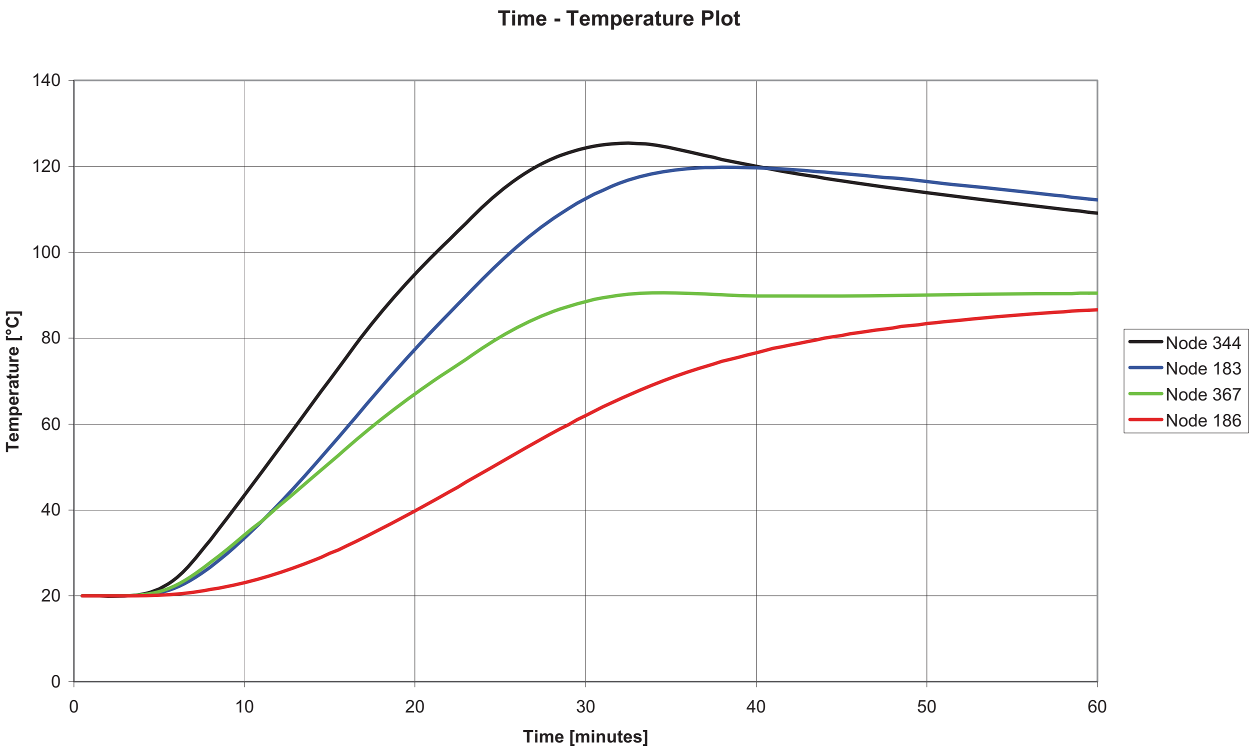


Fig. 33.16 Temperature evolution on the protected diagonal of the lattice girder

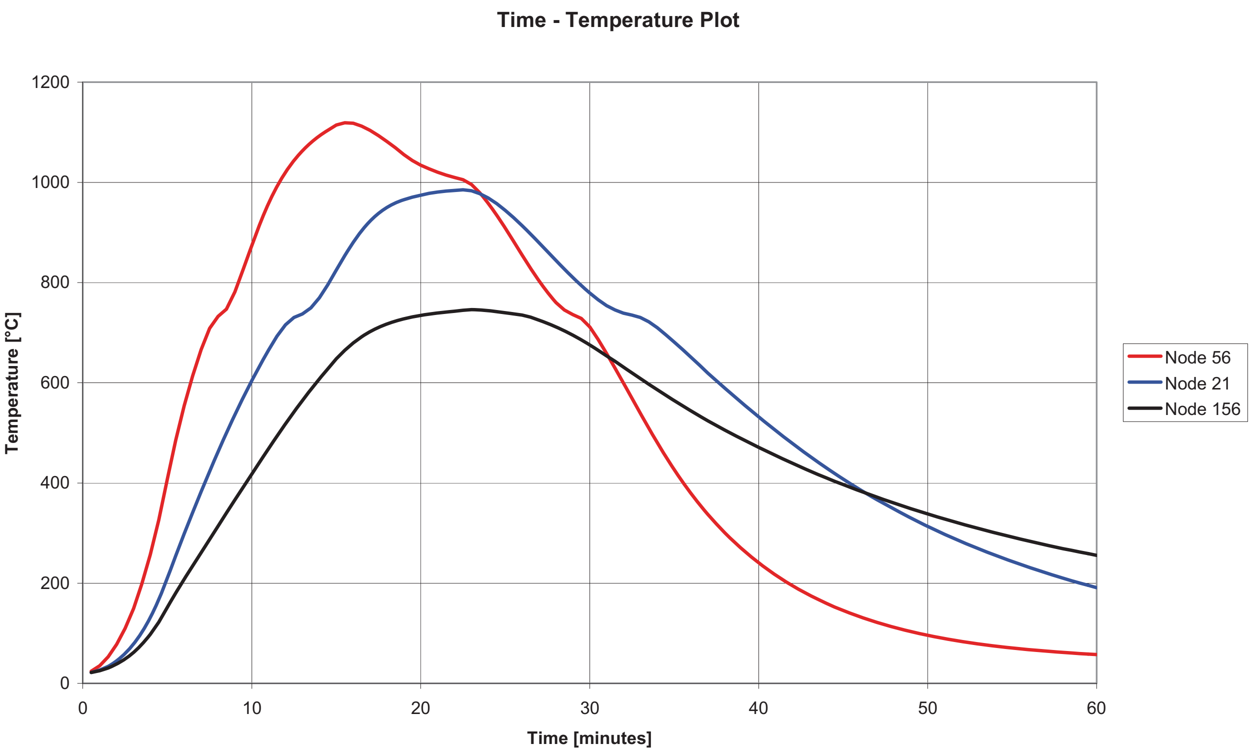


Fig. 33.17 Temperature evolution on the unprotected crossbeam (case A)

The temperature evolution on the unprotected crossbeam (Case A) is shown in Fig. 33.17. The three representative points in which the temperature evolution is checked are selected in the lower flange (Node 21), mid-height of the web (Node 56) and upper flange (Node 156).

In this case, because of the low section factor of the steel girder, the maximum temperature reaches 1119°C after 15.5 minutes (900 sec) in the hottest spot of the cross section (Node 56, which is the mid height of the web), followed by a steep decrease after 22 minutes (1300 seconds).

The temperatures in the upper web are lower than the ones in the rest of the steel, due to the concrete slab which acts as a heat sink. The maximum temperatures in the upper flange do not exceed 750°C.

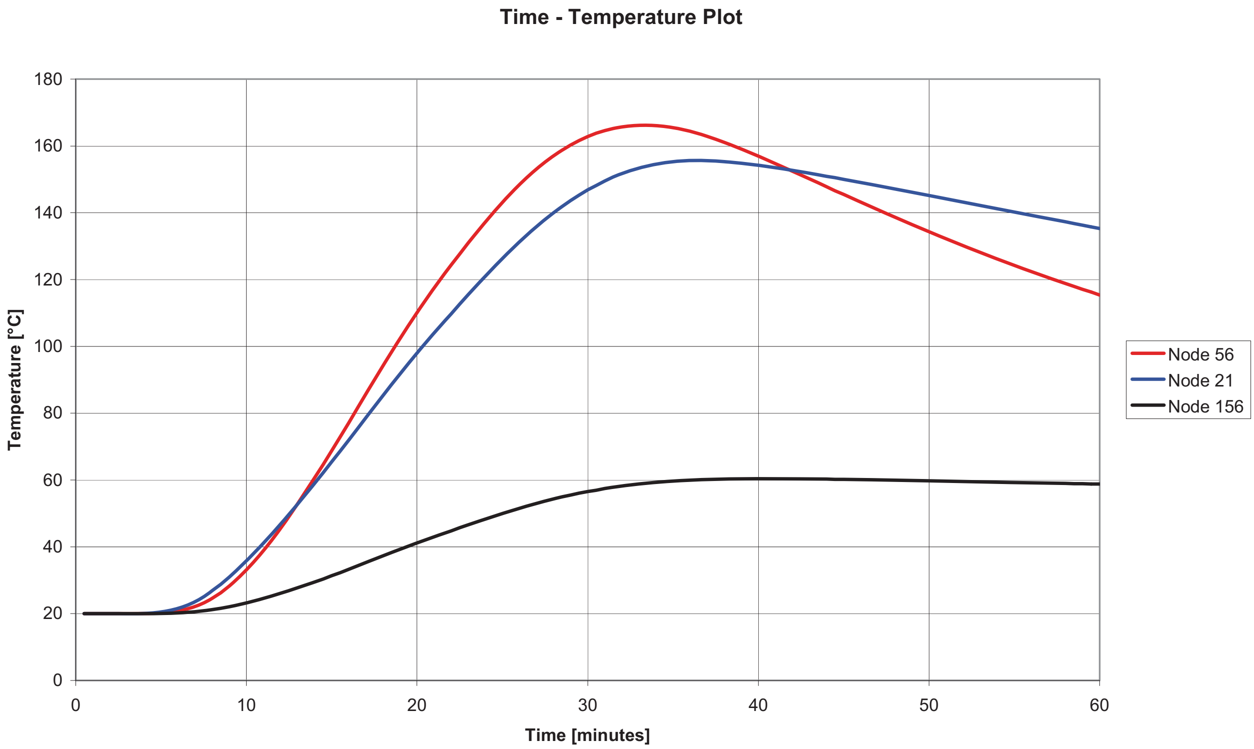


Fig. 33.18 Temperature evolution on the ceiling protected crossbeam (case B)

For the first protection scenario (Case B), using a 15 mm PROMATECT ceiling under the lower flange, the temperature evolution is presented in Fig. 33. 18. The temperature evolution is presented in the same nodes as in the previous case.

The maximum temperature on the cross section of 166.2°C is attained in the node 56 (the mid height of the web) at 33.5 minutes.

In the box protected case (Case C), two extra nodes where the temperature is inspected are added to the set of nodes (see Fig. 33.19).

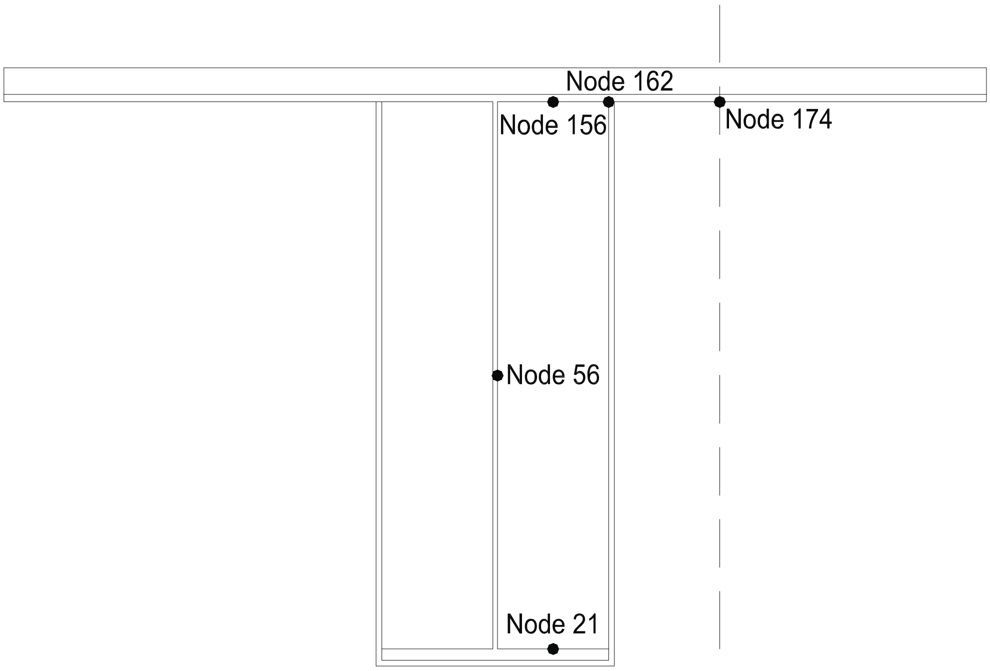


Fig. 33.19 Representative nodes on the box protected cross section of the crossbeam (case C)

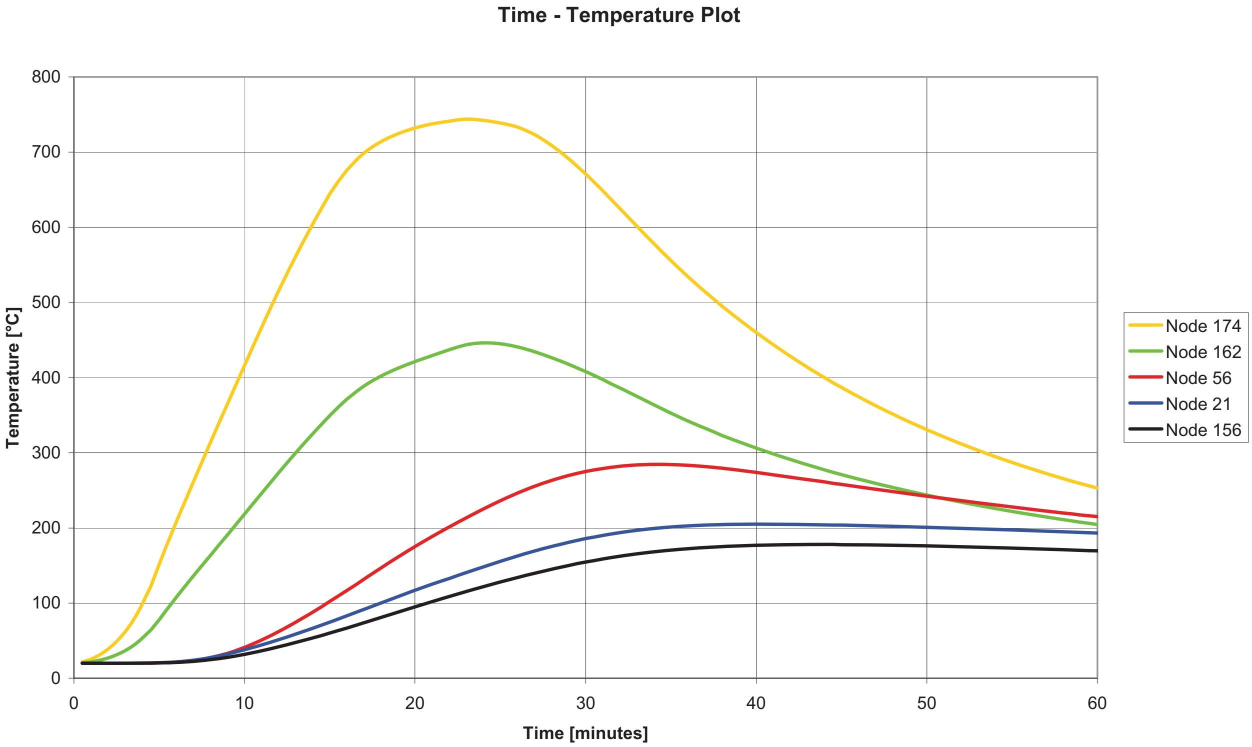


Fig. 33.20 Temperature evolution in the box protected crossbeam (case C)

The node 174 is on the lower edge of the upper flange, at a distance from the node 162 equal to half of the lower flange (≈150 mm). This node is added to inspect the temperatures in the unprotected steel.

While the steel inside the box protection has a temperature of up to 300°C, the bare upper flange, exposed to the fire with a peak of 1100°C reaches at 23 minutes (1400 sec) a temperature of 744°C. The temperature evolution in the five representative nodes of the cross section is presented in Fig. 33.20.

The last protection case (Case D), with a 15 mm PROMATECT box protection around the girder, and continued under the upper flange gives the temperature evolution presented in Fig. 33.21. The same nodes as in the previous model are inspected. With this protection, the maximum temperature in the steel girder at 34 minutes is 274.3°C.

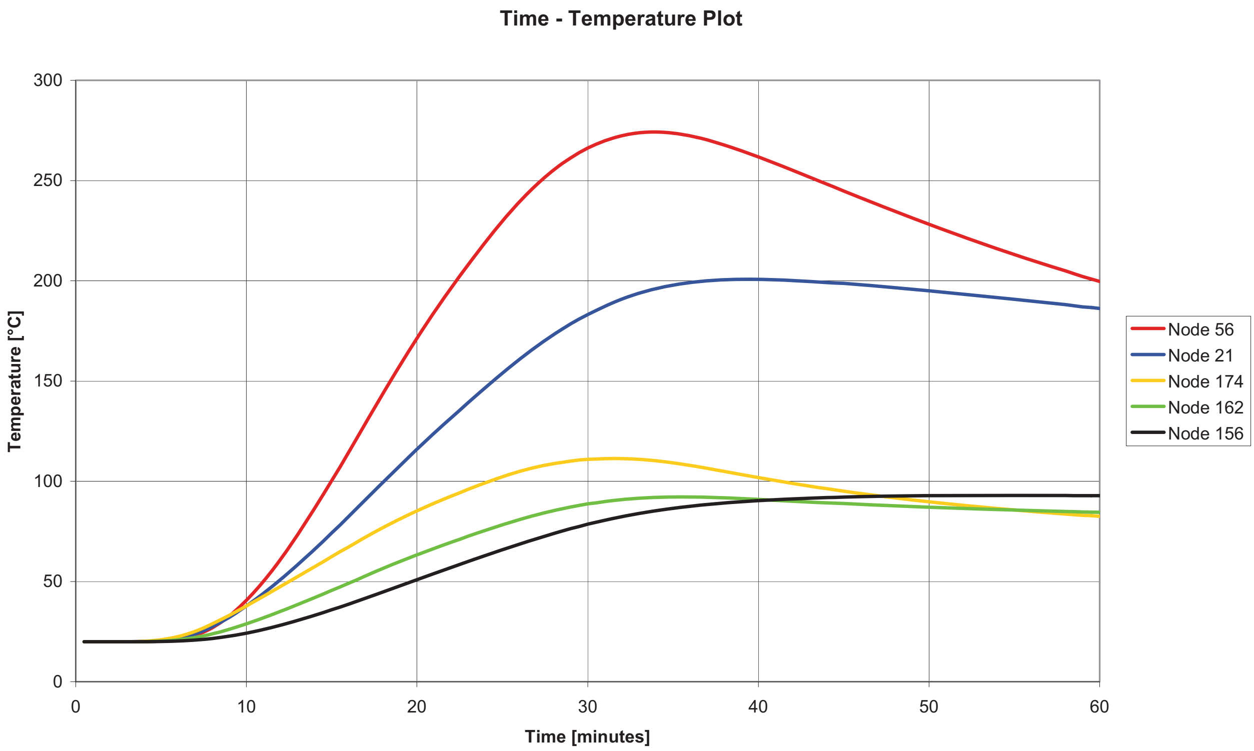


Fig. 33.21 Temperature evolution on the extended box protected crossbeam (case D)

## 33.6 SUMMARY AND CONCLUSIONS

This case study presents the thermal analysis of the main components of a double deck bridge, without any protection and with thermal protection, selecting several protection scenarios to ensure a maximum protection under minimum cost.

Analysing and presenting several cases to the beneficiary gives him the choice of selecting the best cost effective solution to be used.

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