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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.

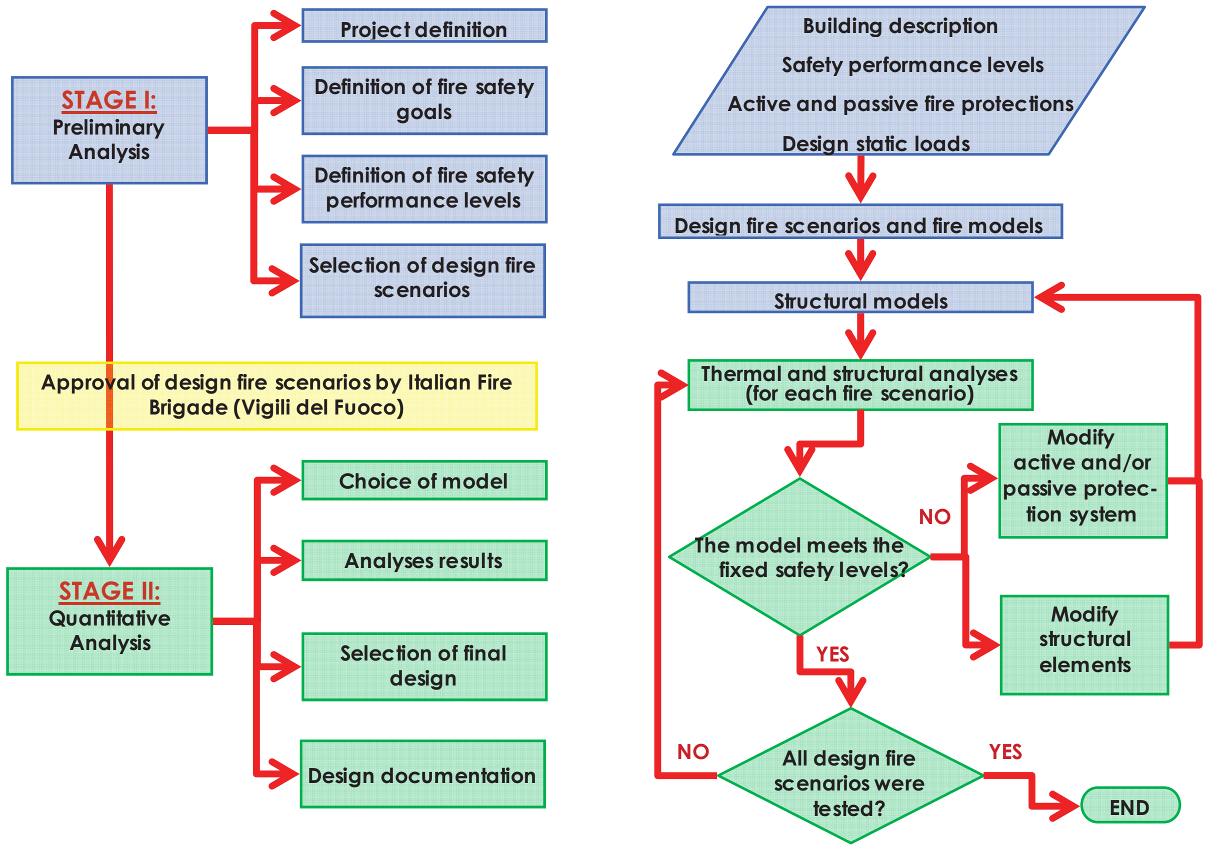
The results of each application of the performance approach to the fire safety should be evaluated through the analysis of the achievement of these objectives.

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

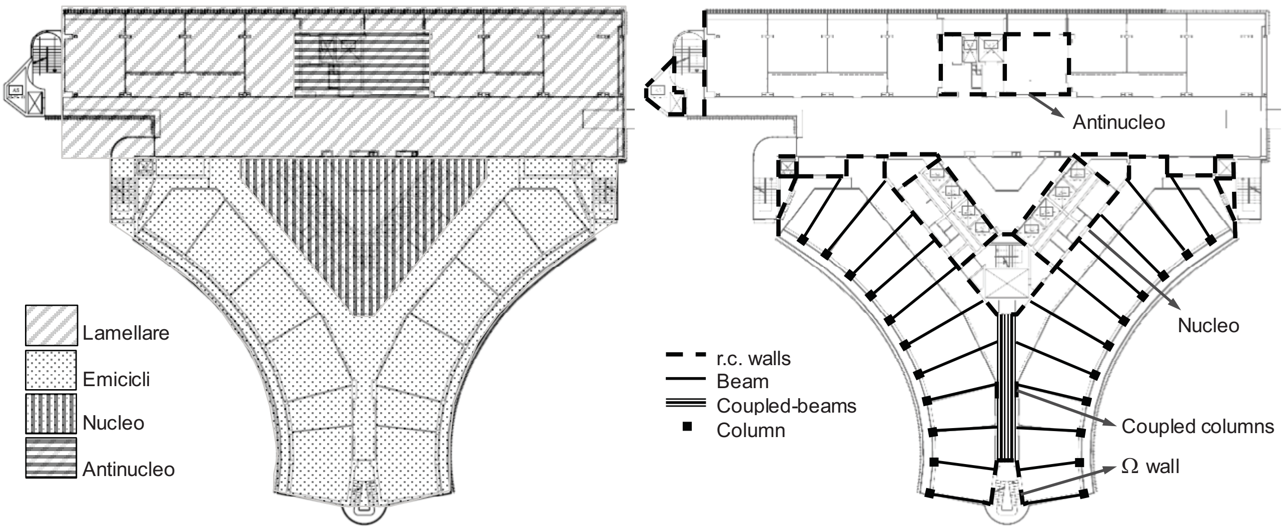
(Tower C), located on the east side, extends from a height of 30.00 m to a height of 69.60 m above sea level; the intermediate one (Tower B) develops from a height of 30.00 m to a height of 89.40 m above sea level, the highest one (Tower A) extends from a height of 30.00 m to a height of 112.50 m above 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.

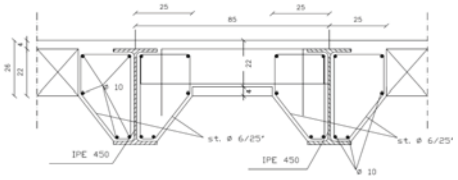
Fig. 1.2 New Courthouse of Naples: South side view



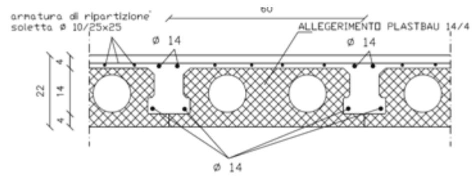
(a) (b)

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

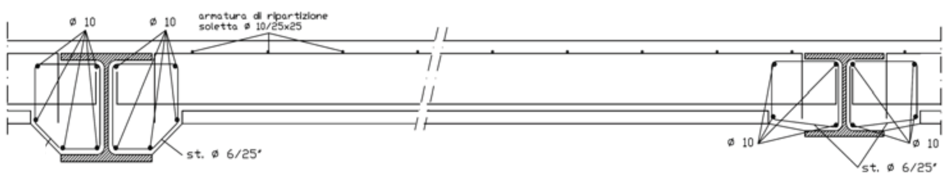
Referring to “emicicli” zone (see Fig. 1.3a), from 30.00 m above sea level, there are primary steel beams arranged in a radial pattern, which join the exterior steel columns to the reinforced concrete wall of the “nucleo” zone or to the coupled beams (which join the “Ω wall“ to the “nucleo” zone wall, 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.



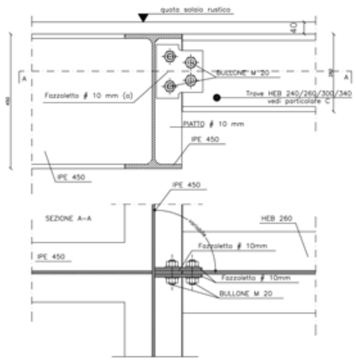
a) cross-section of coupled beams



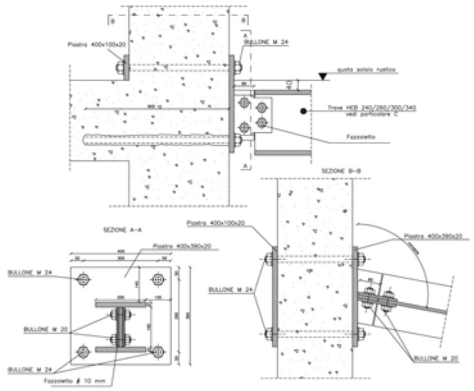
b) cross-section of reinforced concrete slab



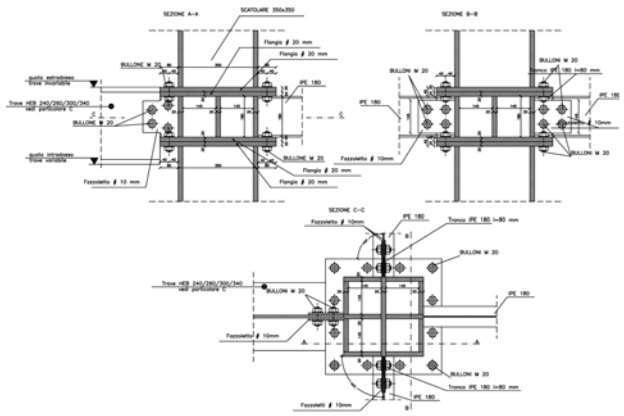
c) cross-section of primary beam and longitudinal-section of reinforced concrete slab



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.

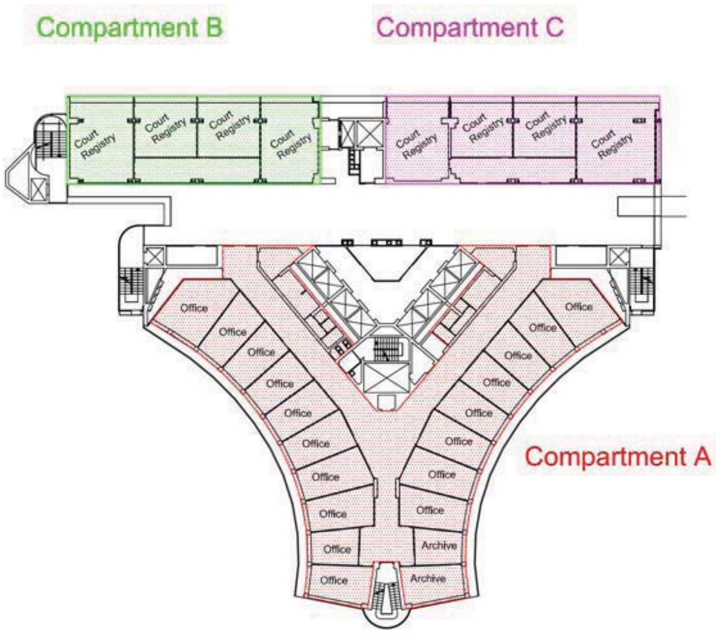


Fig. 1.5 Fire Compartments

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.

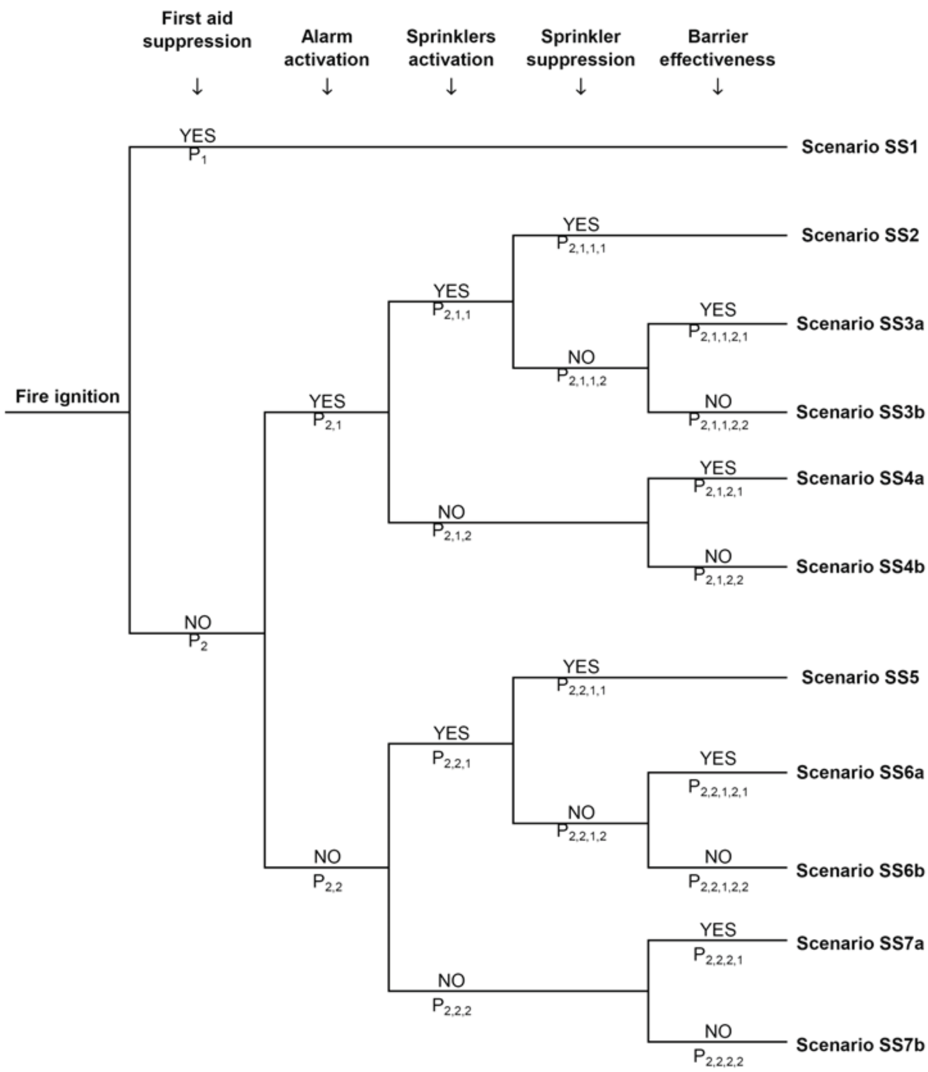


Fig. 1.6 Event tree

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):

$$R = P \cdot C \tag{2}$$

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.

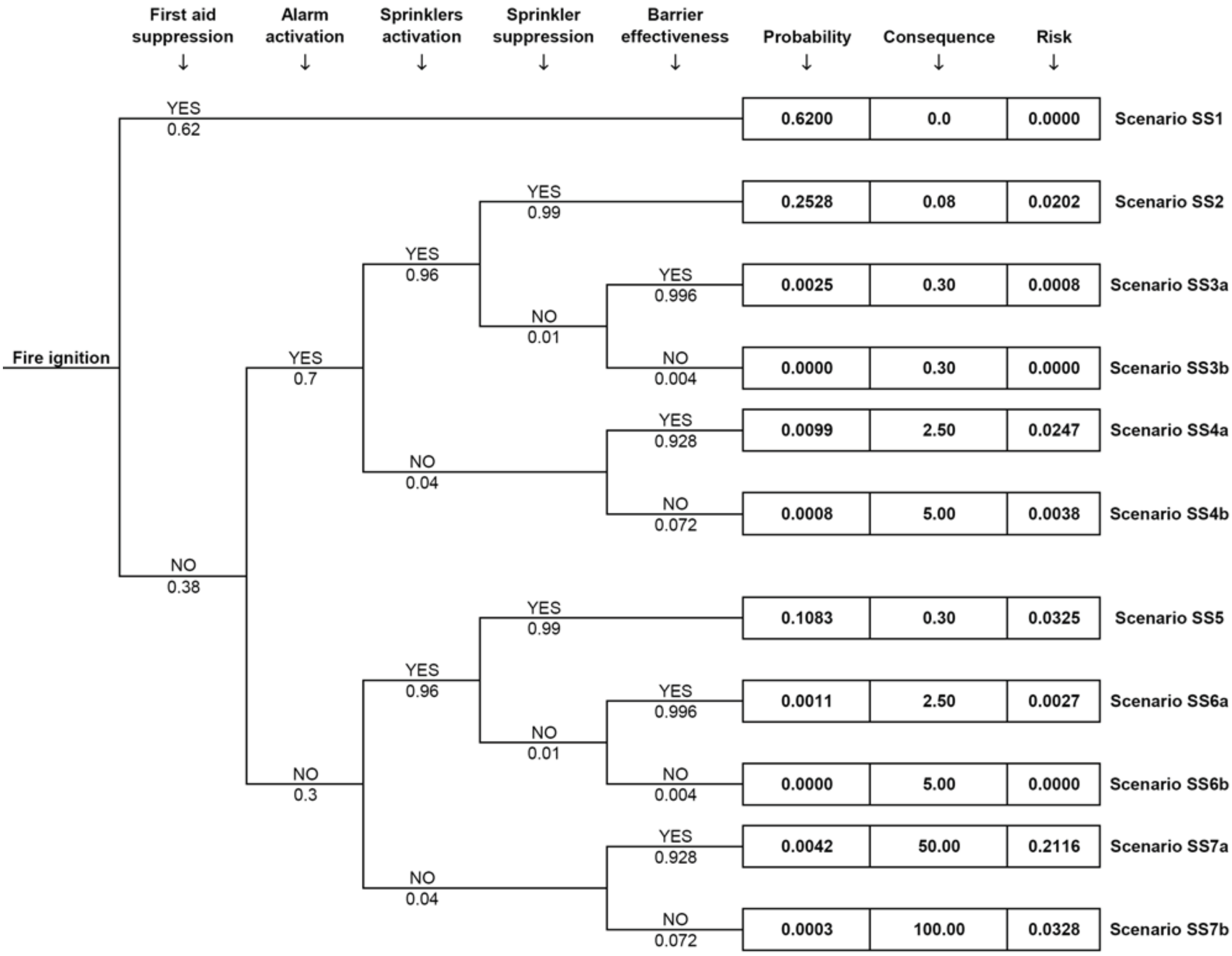


Fig. 1.7 Event tree

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.

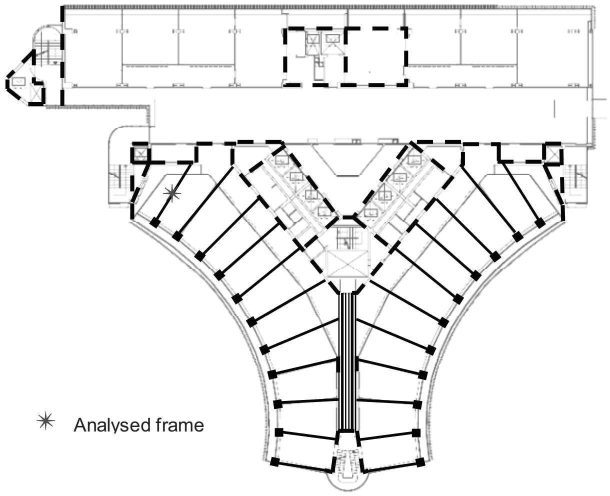


Fig. 1.8 The analysed frame

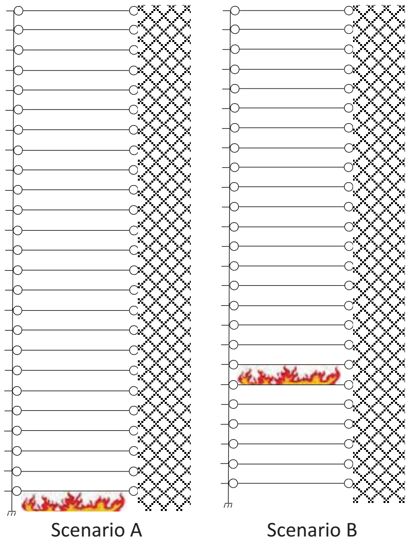


Fig. 1.9 Fire scenarios: possible position along height

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.

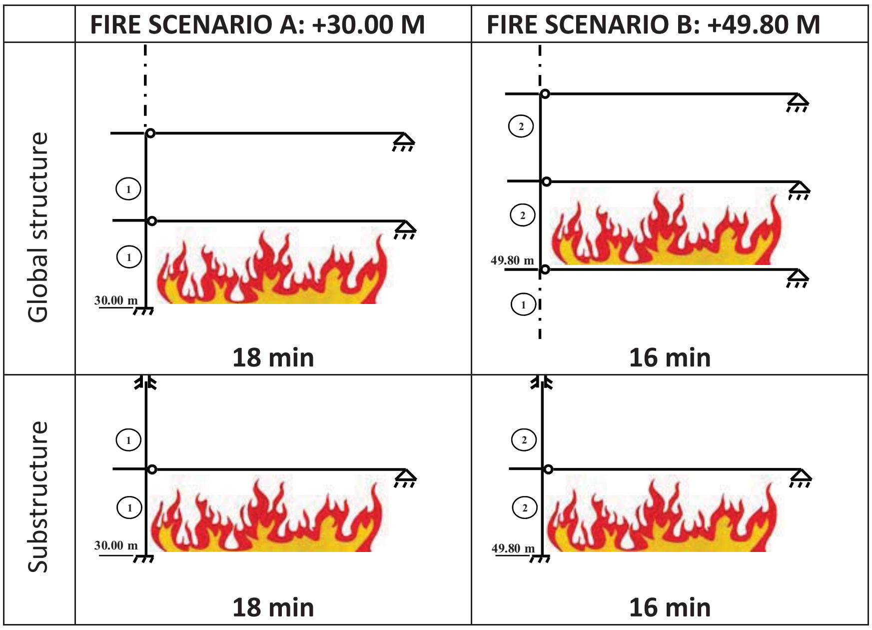


Fig. 1.10 Analyses results

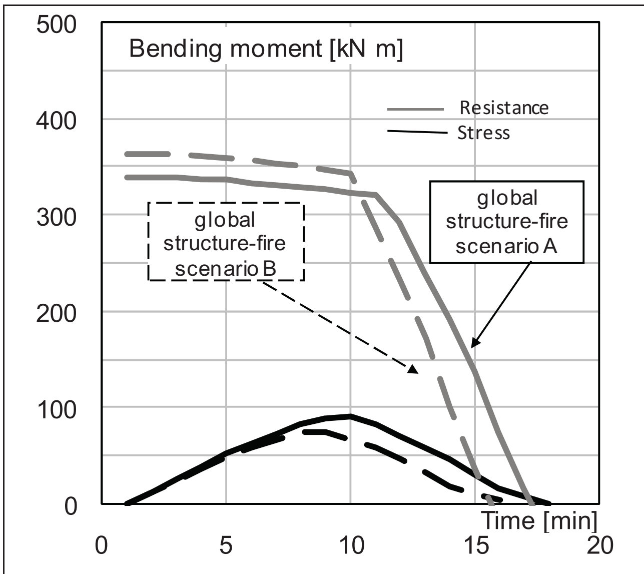


Fig. 1.11 Acting bending moment and Resistance capacity to combined compression and flexure on the heated column of global structure – comparison between fire scenario A and B.

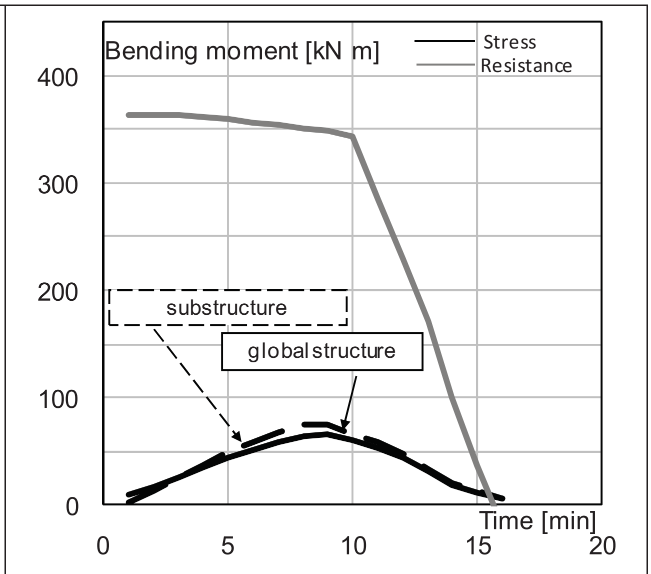


Fig. 1.12 Comparison between global structure and substructure results – Fire scenario B.

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).

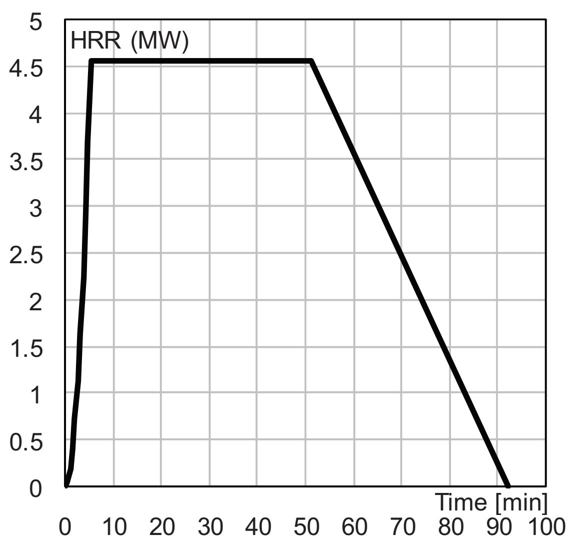


Fig. 1. 13 Rate of Heat Release

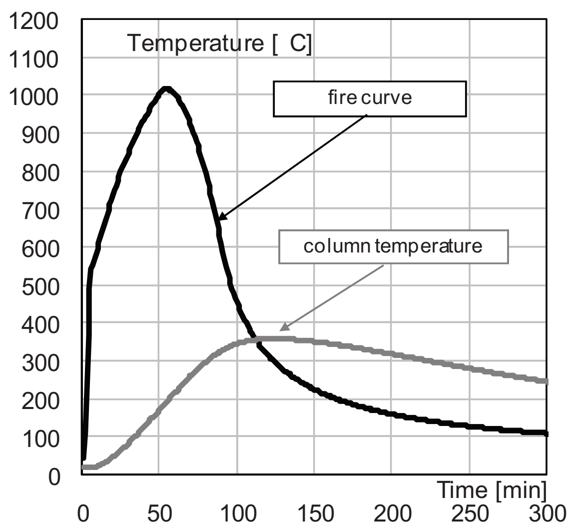


Fig. 1. 14 Comparison between fire curve and column temperature

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).

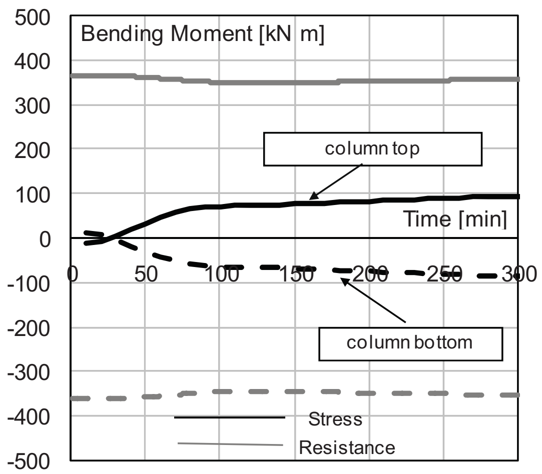


Fig. 1. 15 Bending moments and Resistance capacity to combined compression and flexure on heated column

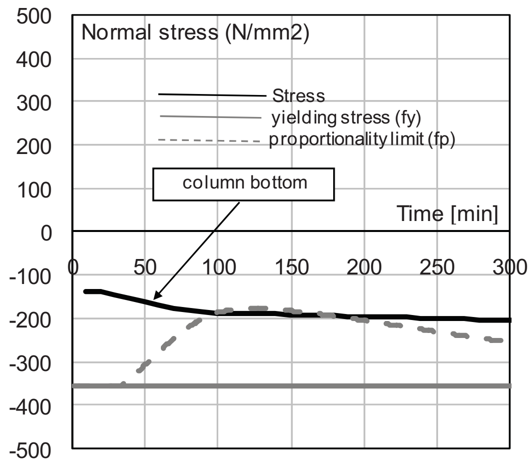


Fig. 1. 16 Comparison between normal stress, proportionality limit and yielding stress in heated column

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.

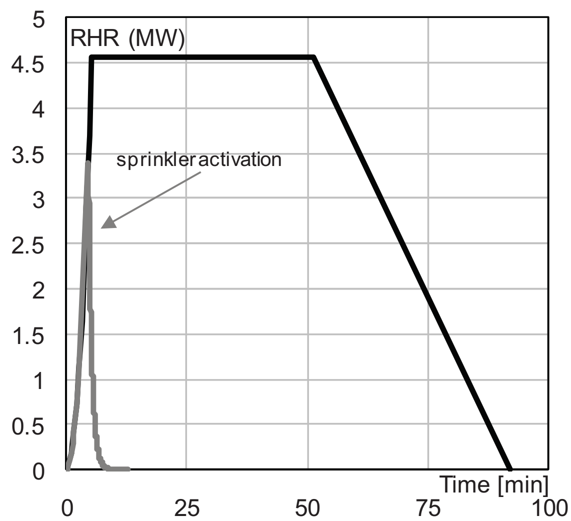


Fig. 1.17 RHR curves

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.

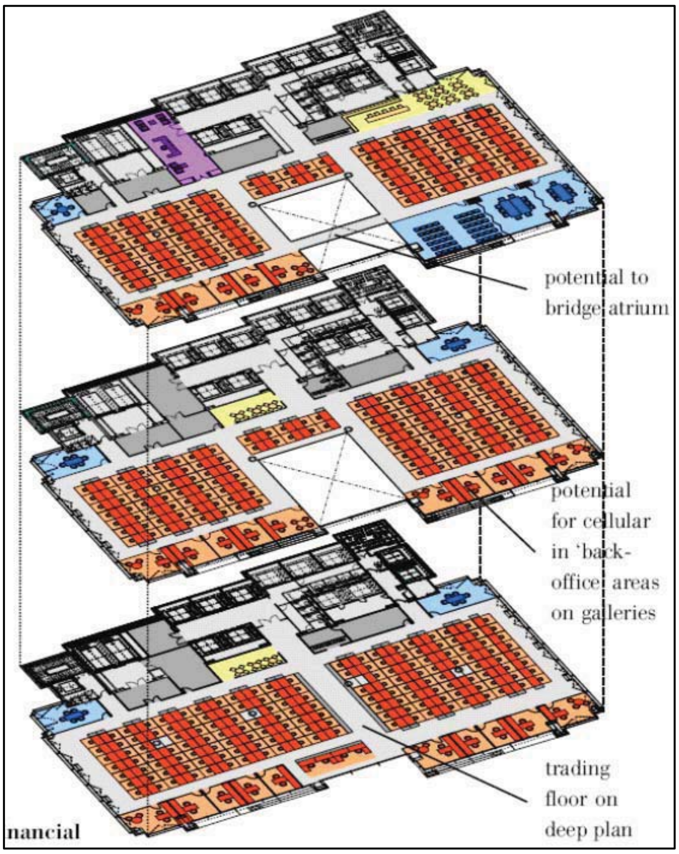

Fig. 2.2 Layout of a three-floor "village"

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

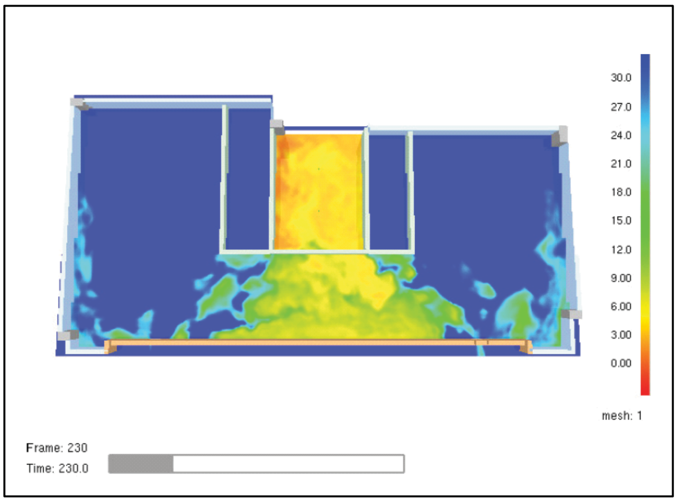

Fig. 2.3 CFD model of smoke spread from atrium to upper floor of a village

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.

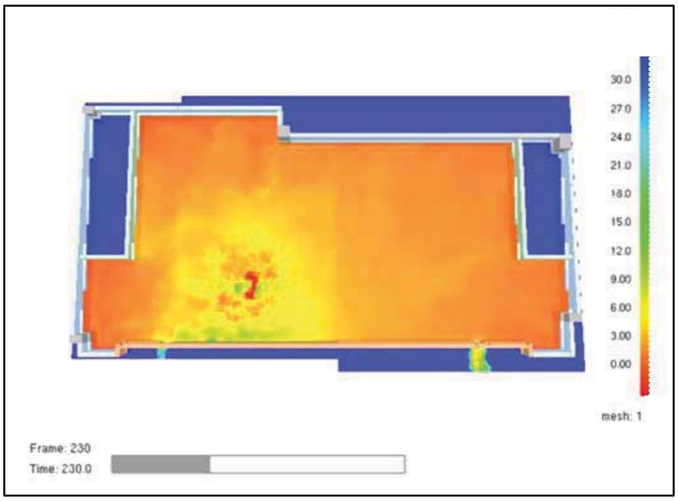

Fig. 2.4 CFD model of smoke conditions on a floor without an atrium

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.

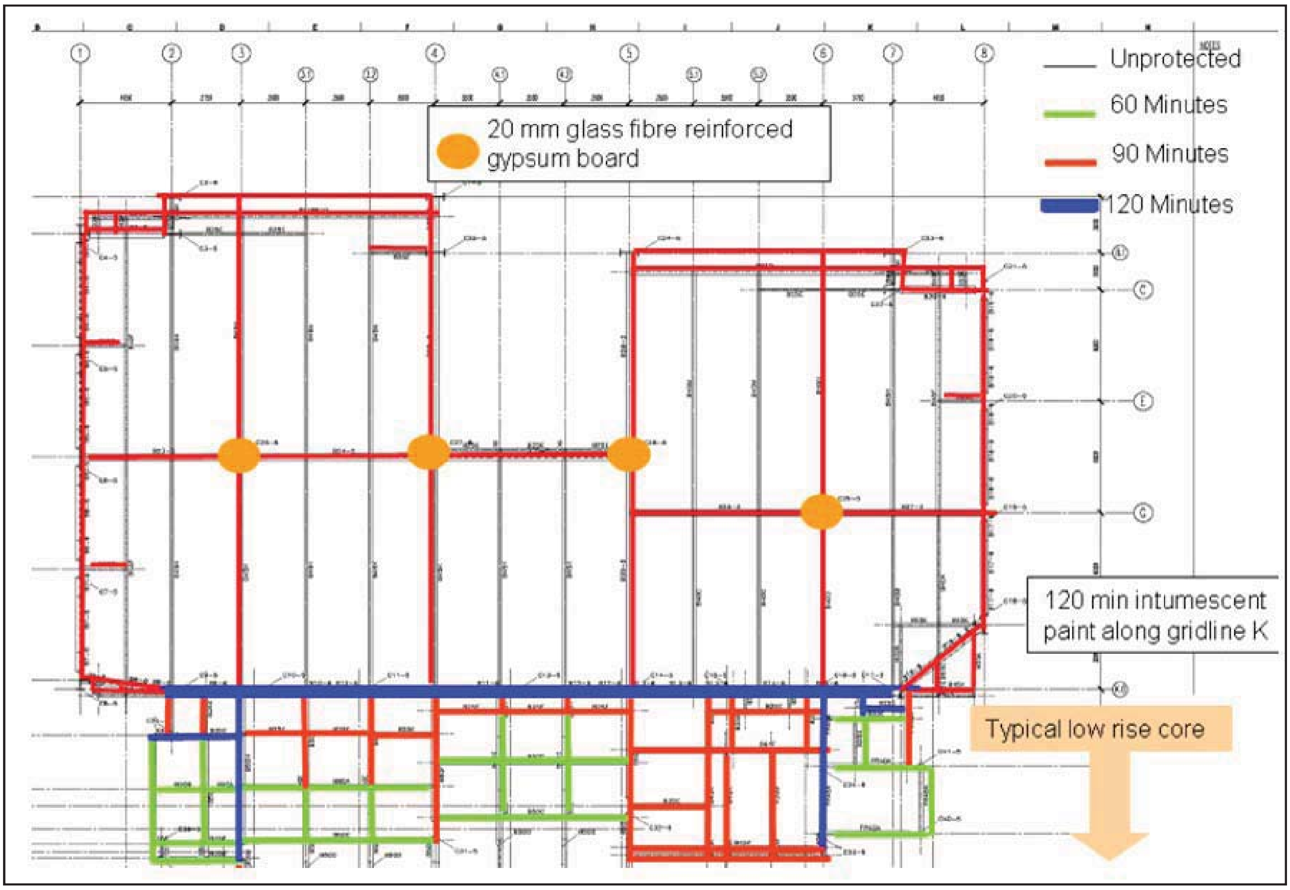

Fig. 2.5 Structural floor layout

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.

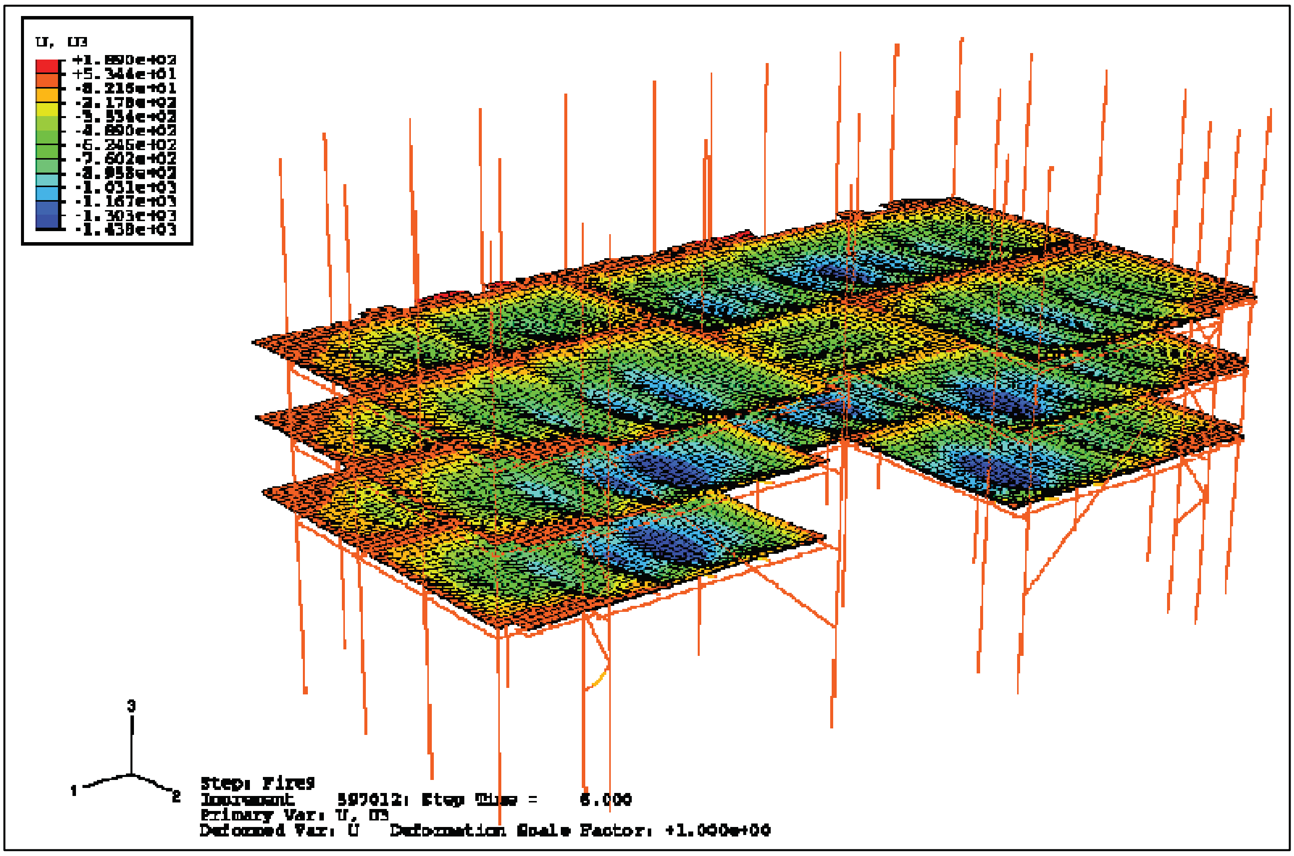

Fig.2.6 ABAQUS thermo-structural model of a full floor, showing deflection contours

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.

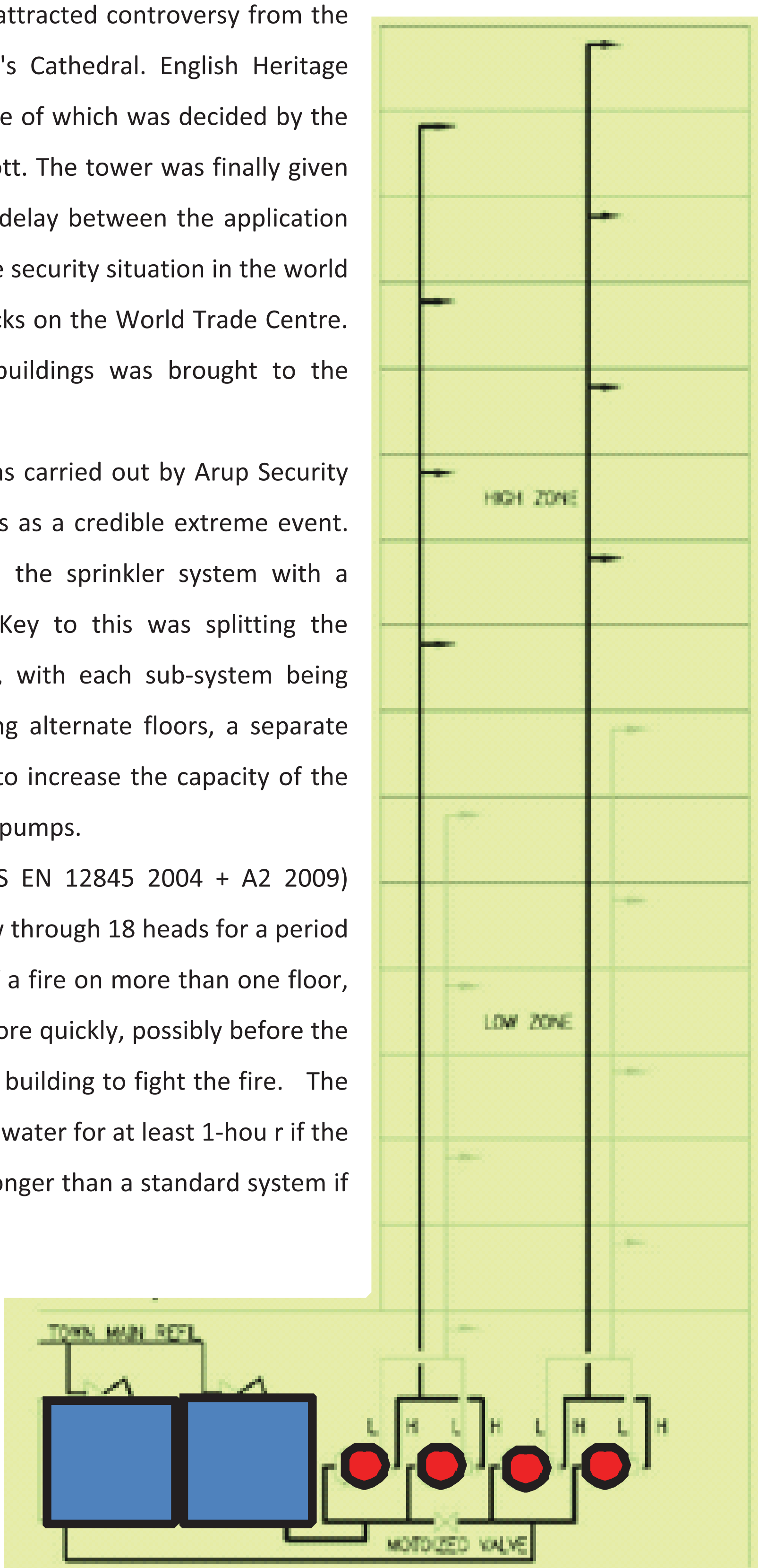

Fig. 2.7 Enhanced sprinkler system layout

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 1st 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.

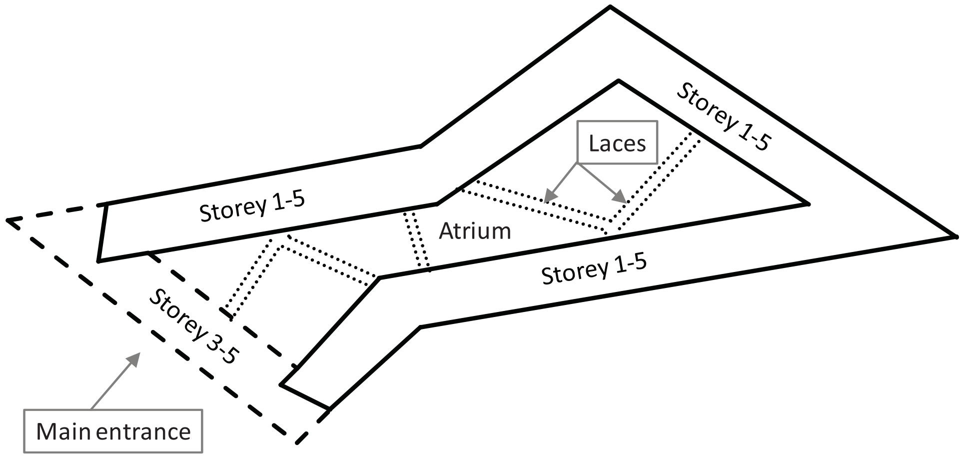

Fig. 3.1 Sketch of adidas-Laces

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.

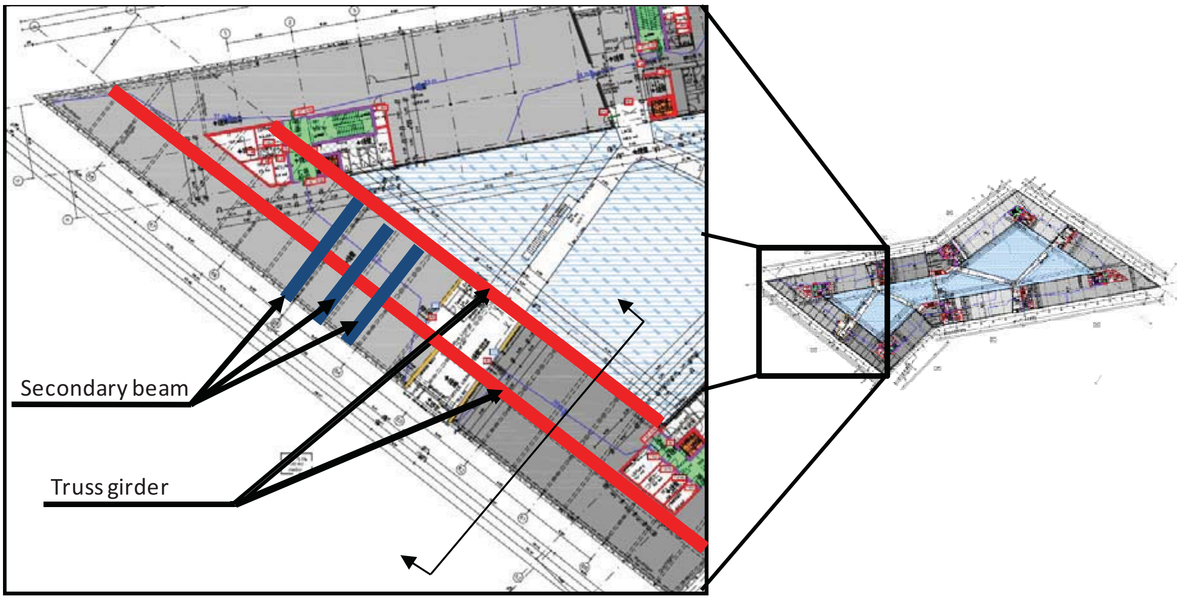

Fig. 3.4 Location of both trusses and some of the secondary beams

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_α) 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.

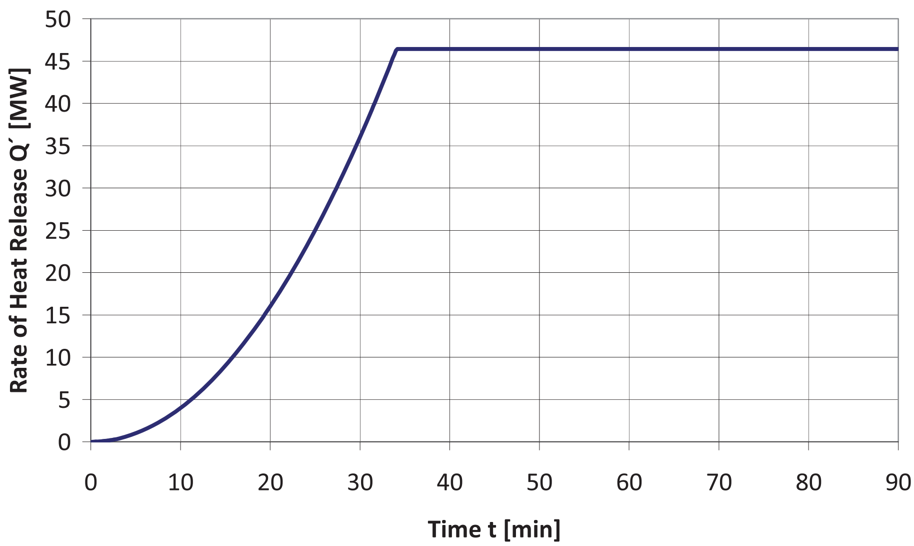

Fig. 3.5 Rate of heat release of design fire

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.

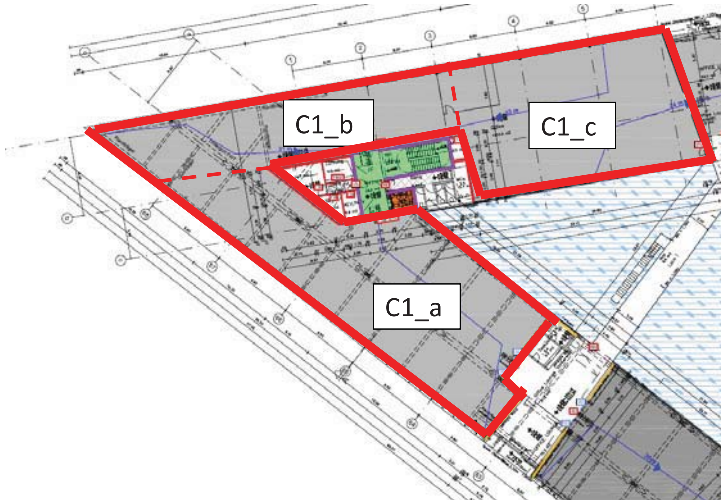

Fig. 3.6 Definition of compartments for multi-room zone-model-analysis

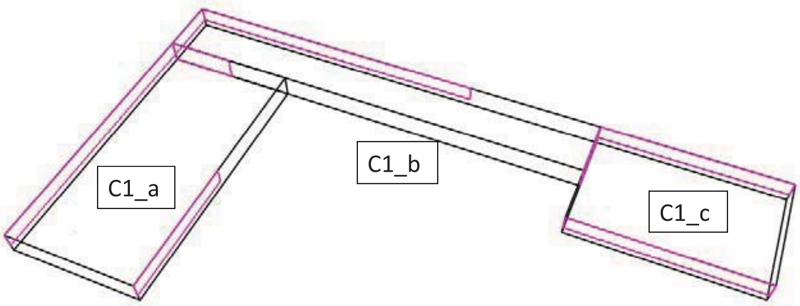

Fig. 3.7 Visualization of compartments in multi-room zone-model-analysis with CFAST

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.

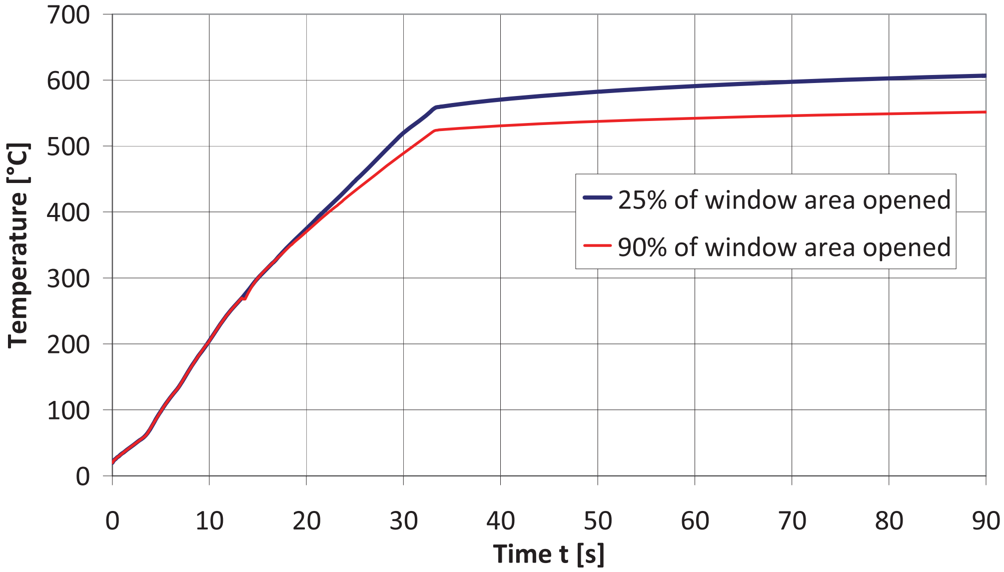

Fig. 3.8 Compartment temperatures with different opening factors

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.

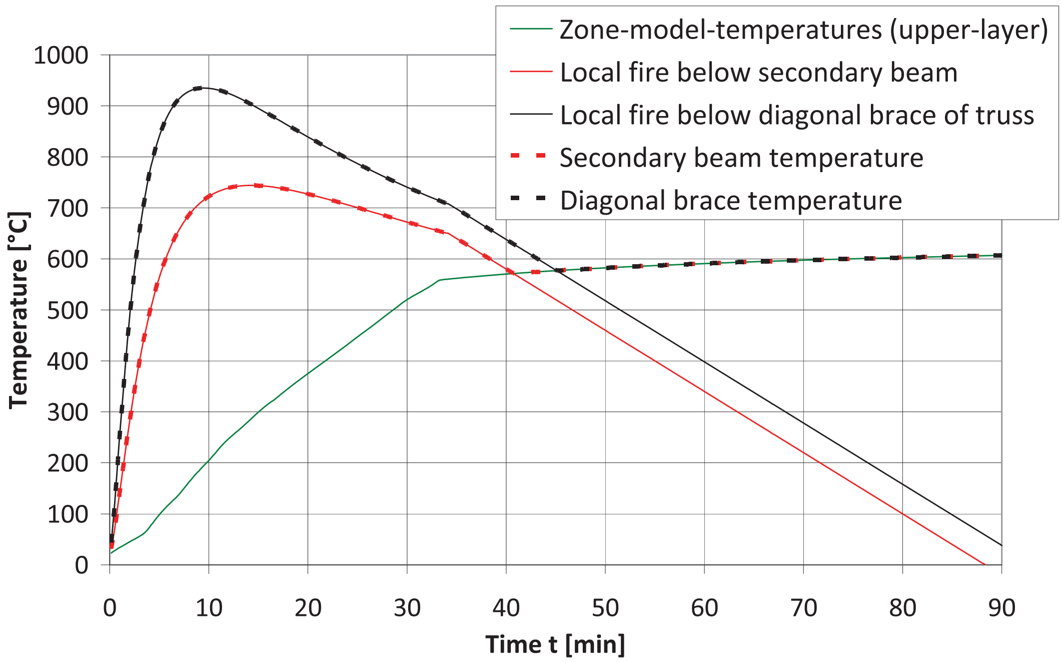

Fig. 3.9 Decisive temperatures for thermal analysis of structural members

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).

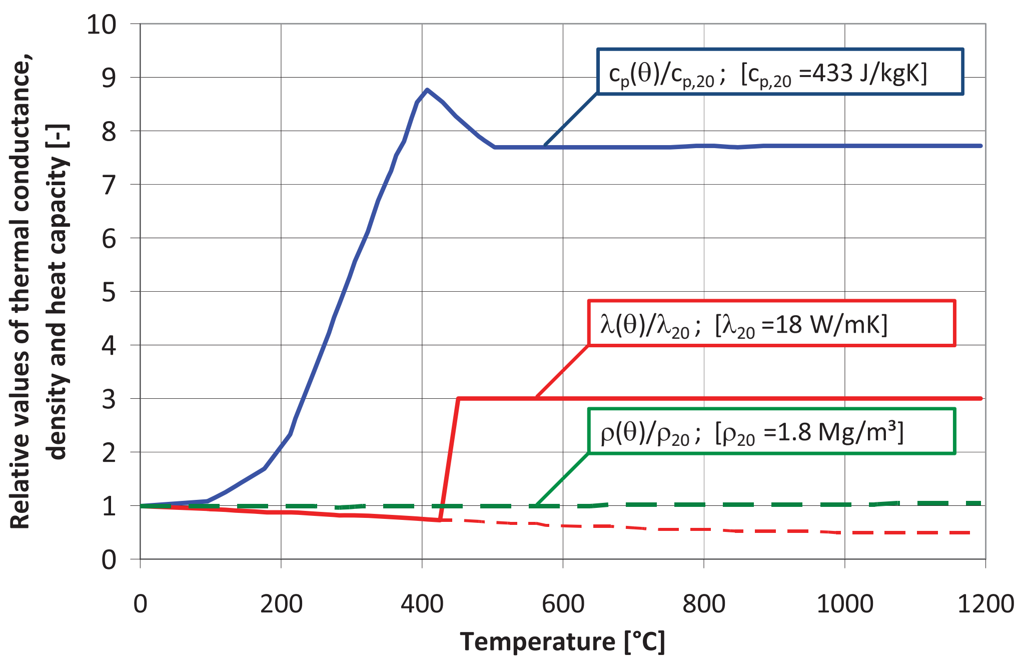

Fig. 3.10 Material properties for intumescent coatings according to Dorn, 2003

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.

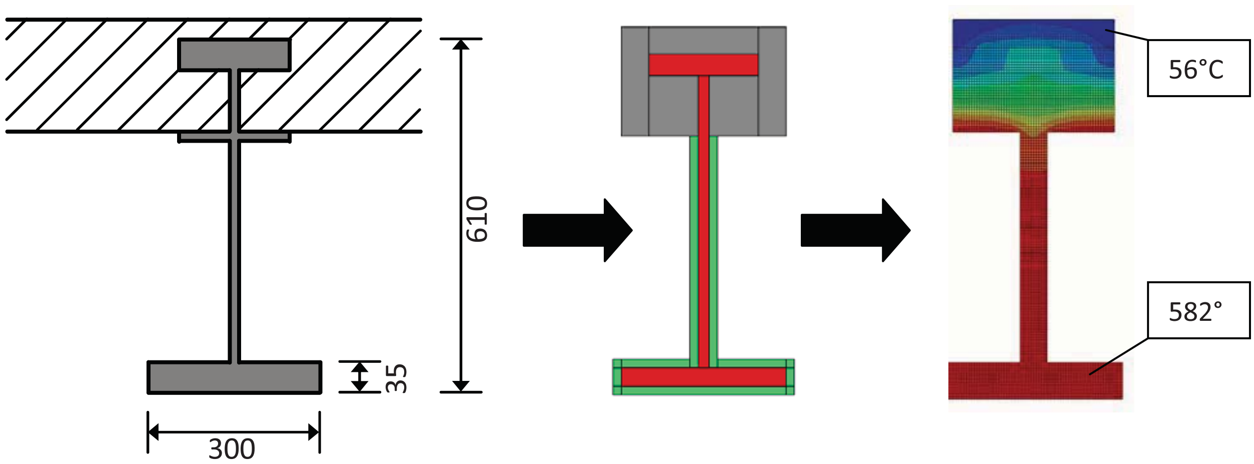

Fig. 3.11 Geometry of secondary beam, numerical model geometry and temperature field at t=90 min

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.

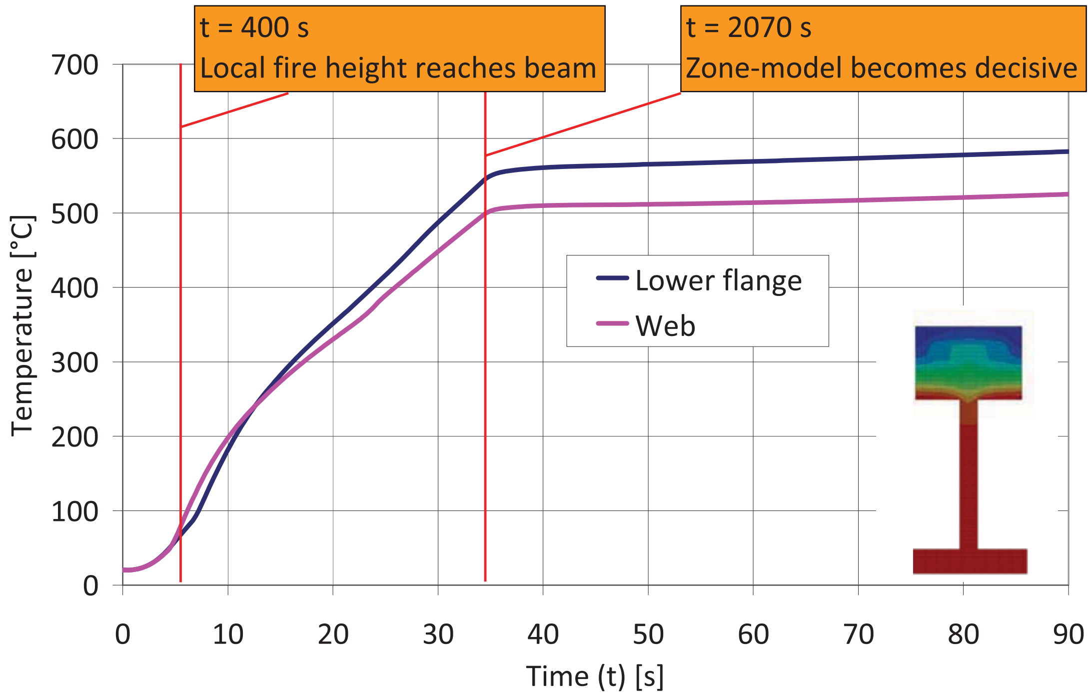

Fig. 3.12 Average temperatures of lower flange and web of secondary beam during design fire

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.

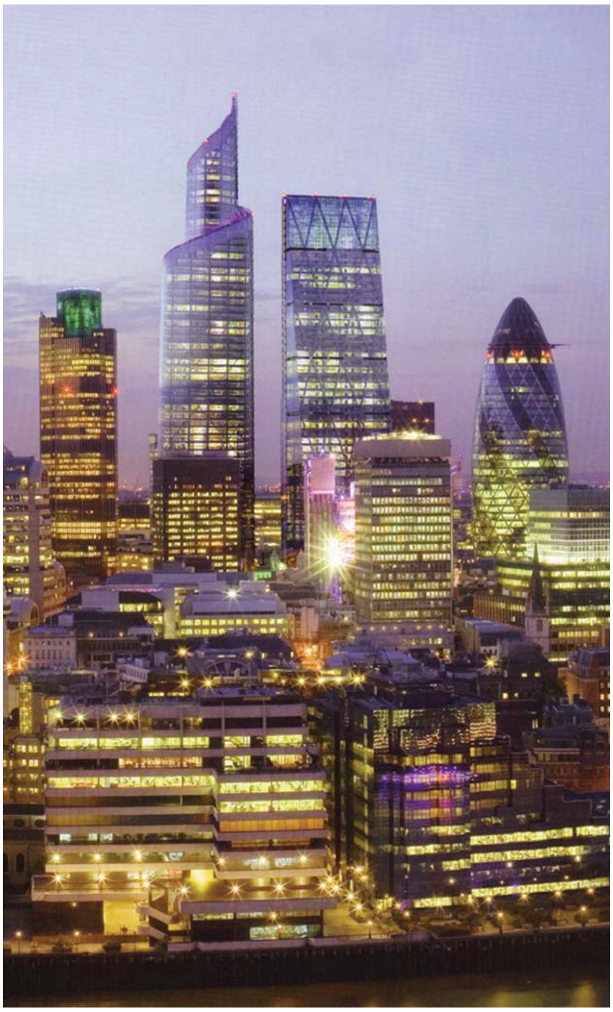

Fig. 4.1 The Pinnacle as part of the City of London skyline - visualization

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.

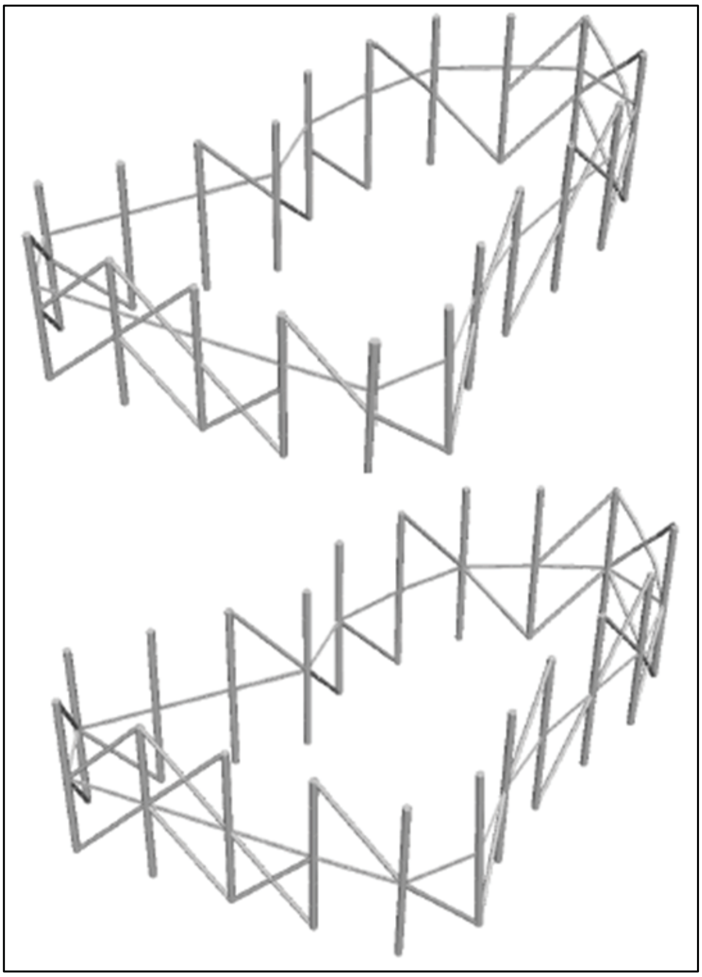

Fig. 4.2 Bracing arrangement details

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.

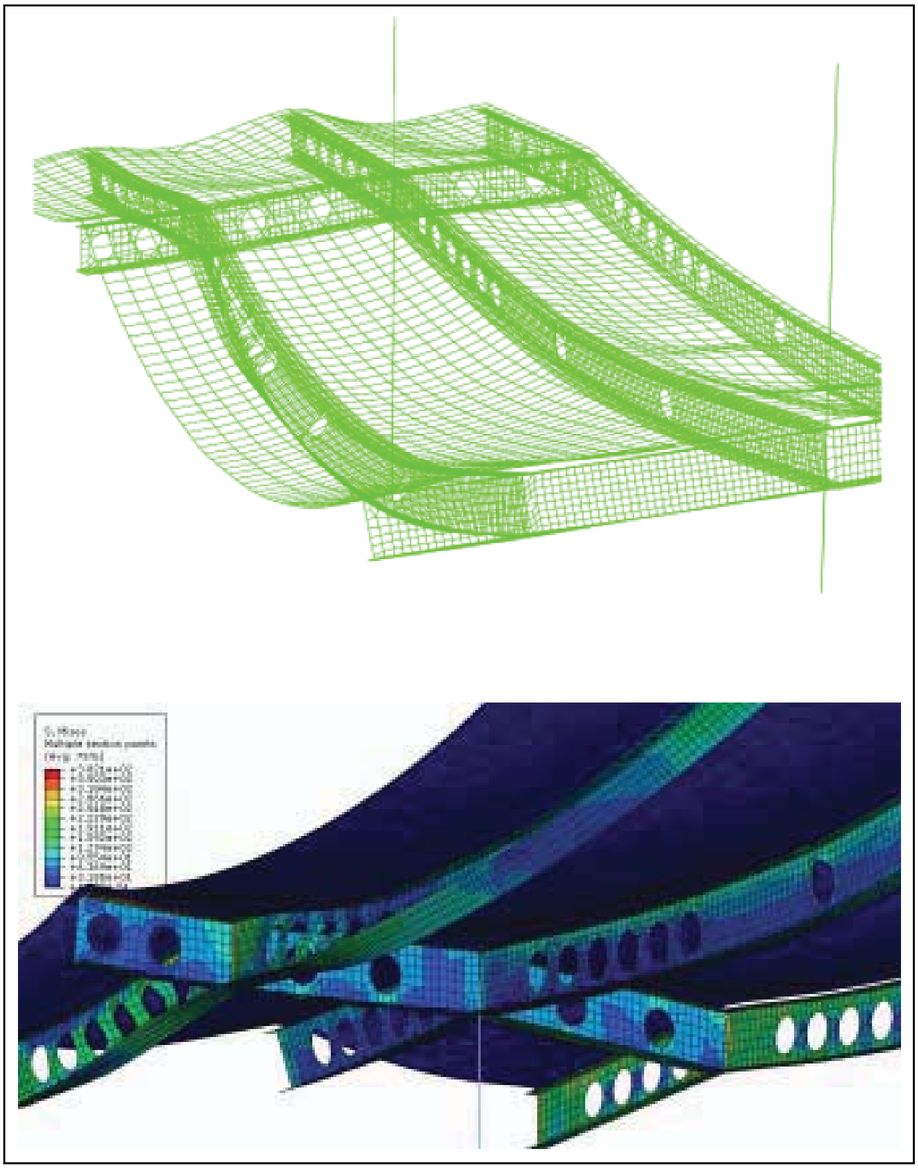

Fig. 4.3 ABAQUS models observing localised behaviour of composite floor employing cellular steel downstand beams

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.

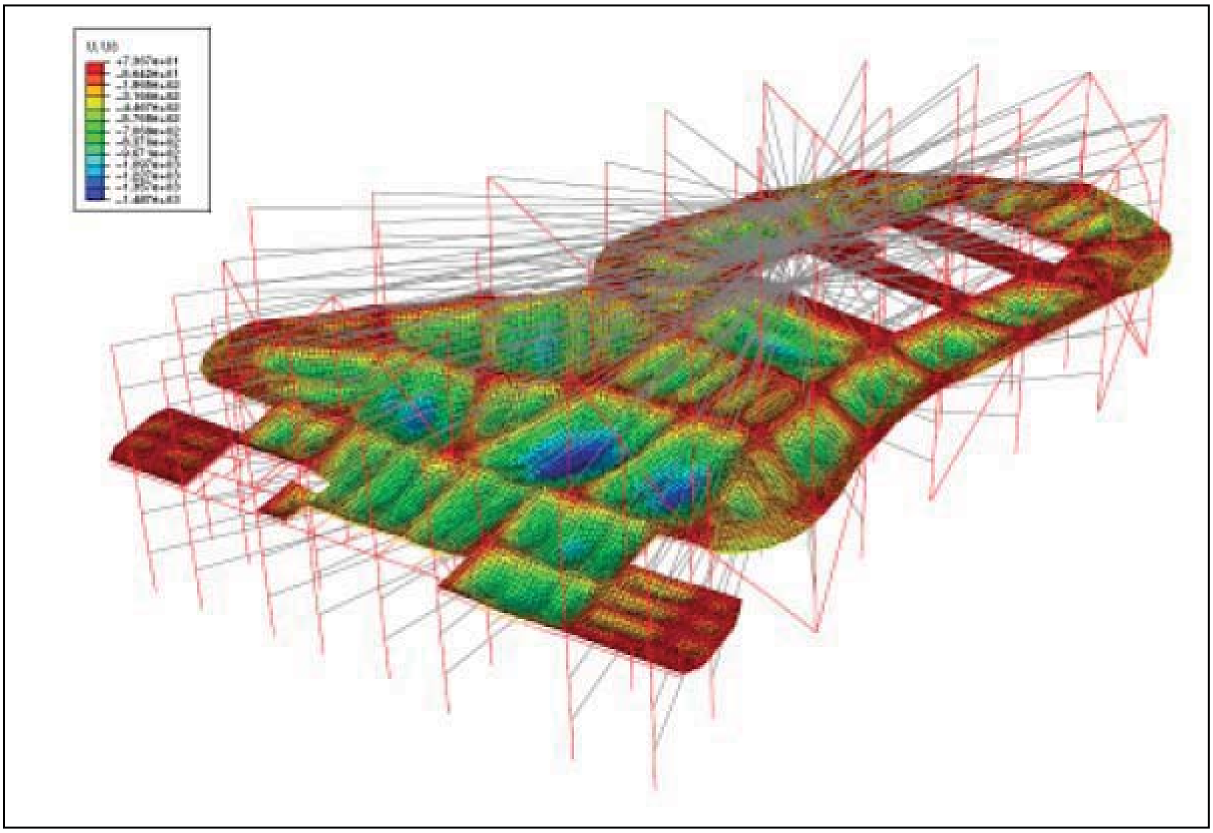

Fig. 4.4 ABAQUS model of one complete composite floor plate

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.

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.

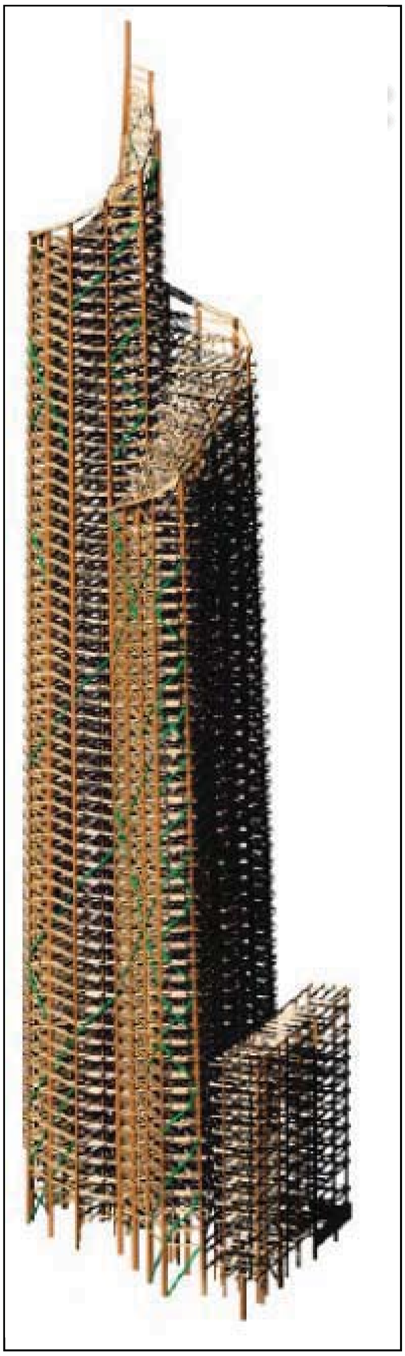

Fig. 4.5 Full-structure frame model

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.

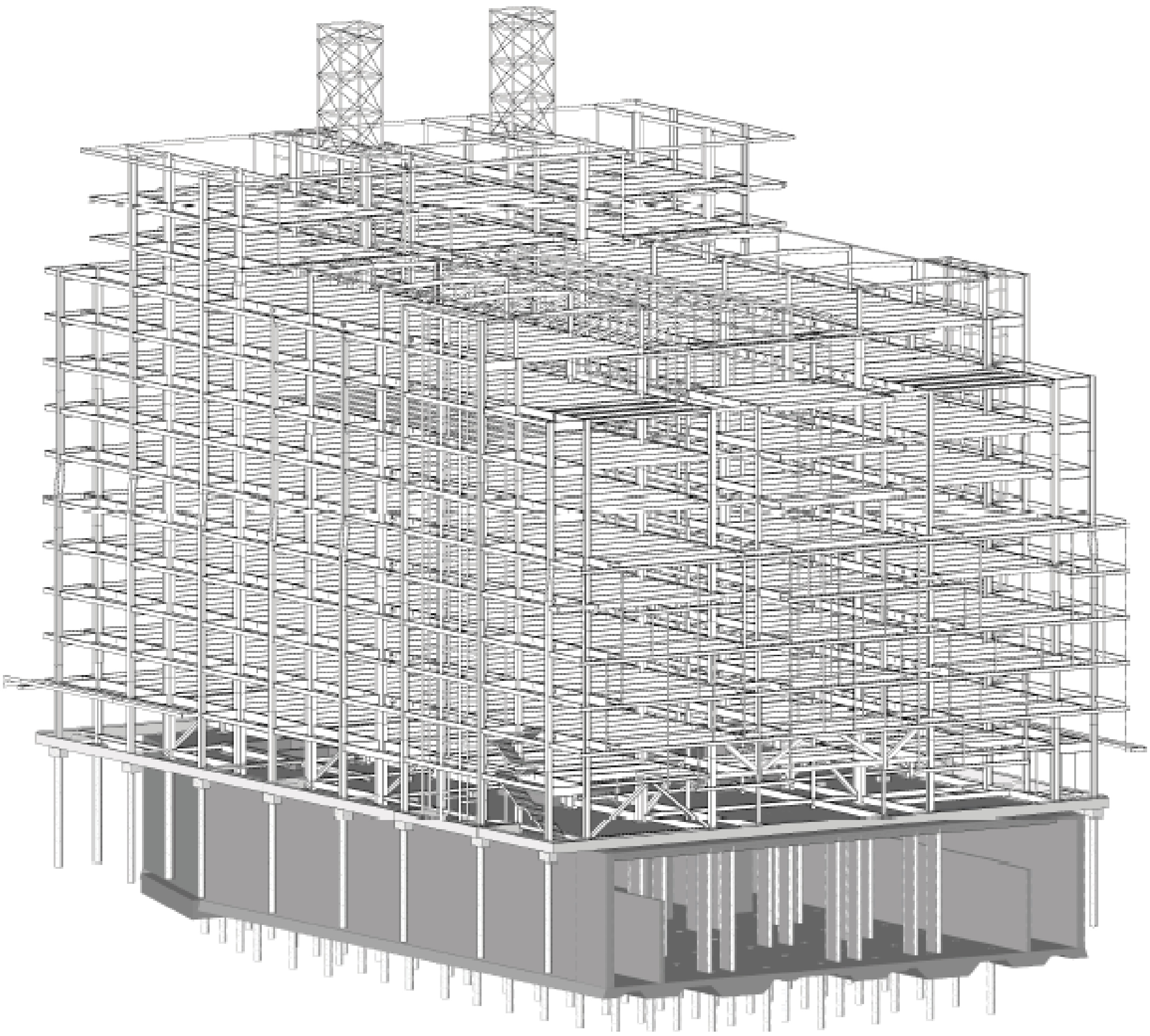

Fig. 5.1 Isometric view of Britomart East building

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".

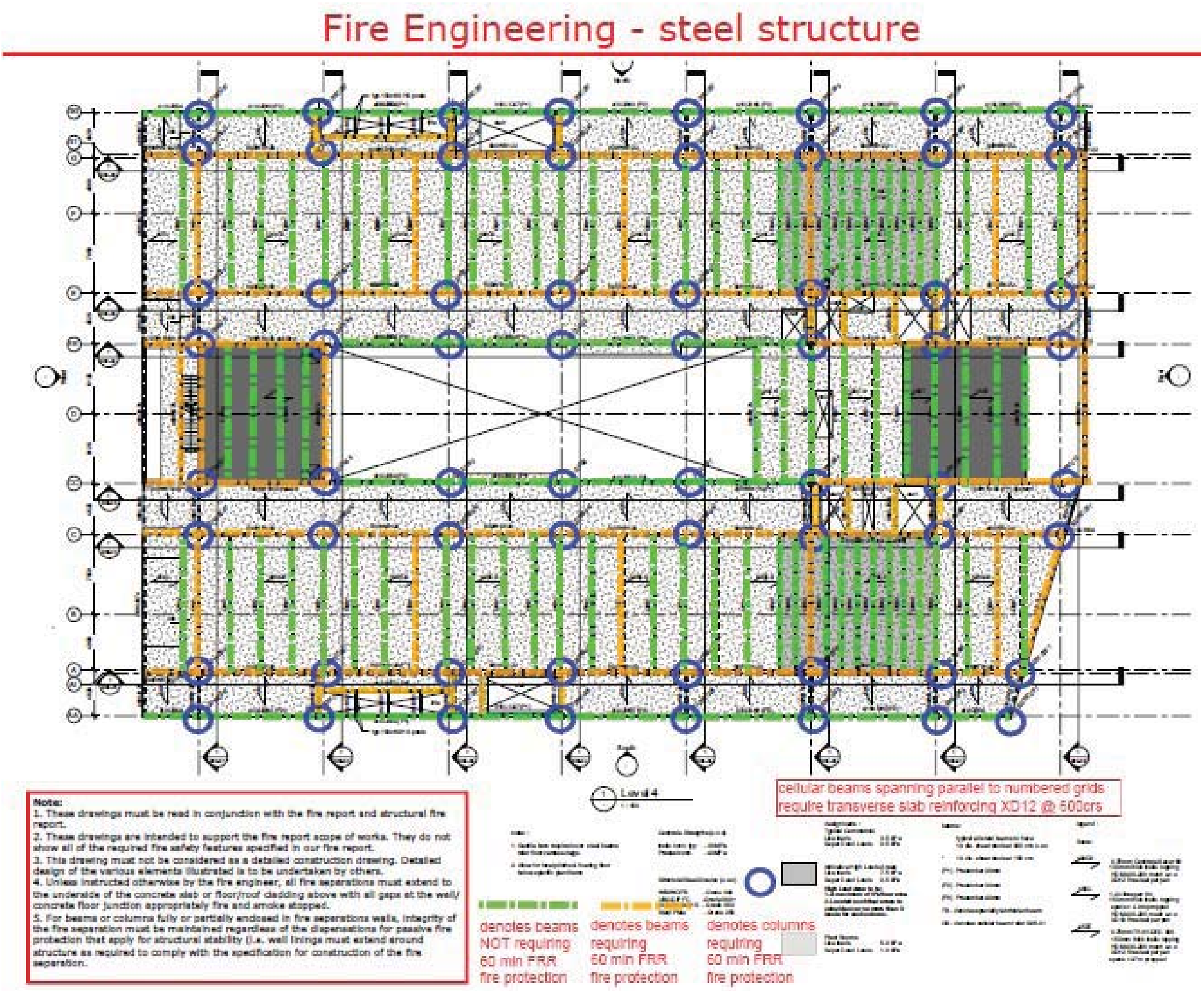

Fig. 5.2 General arrangement of typical floor showing passive fire protection to steel beams

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 1.2×10^{-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.

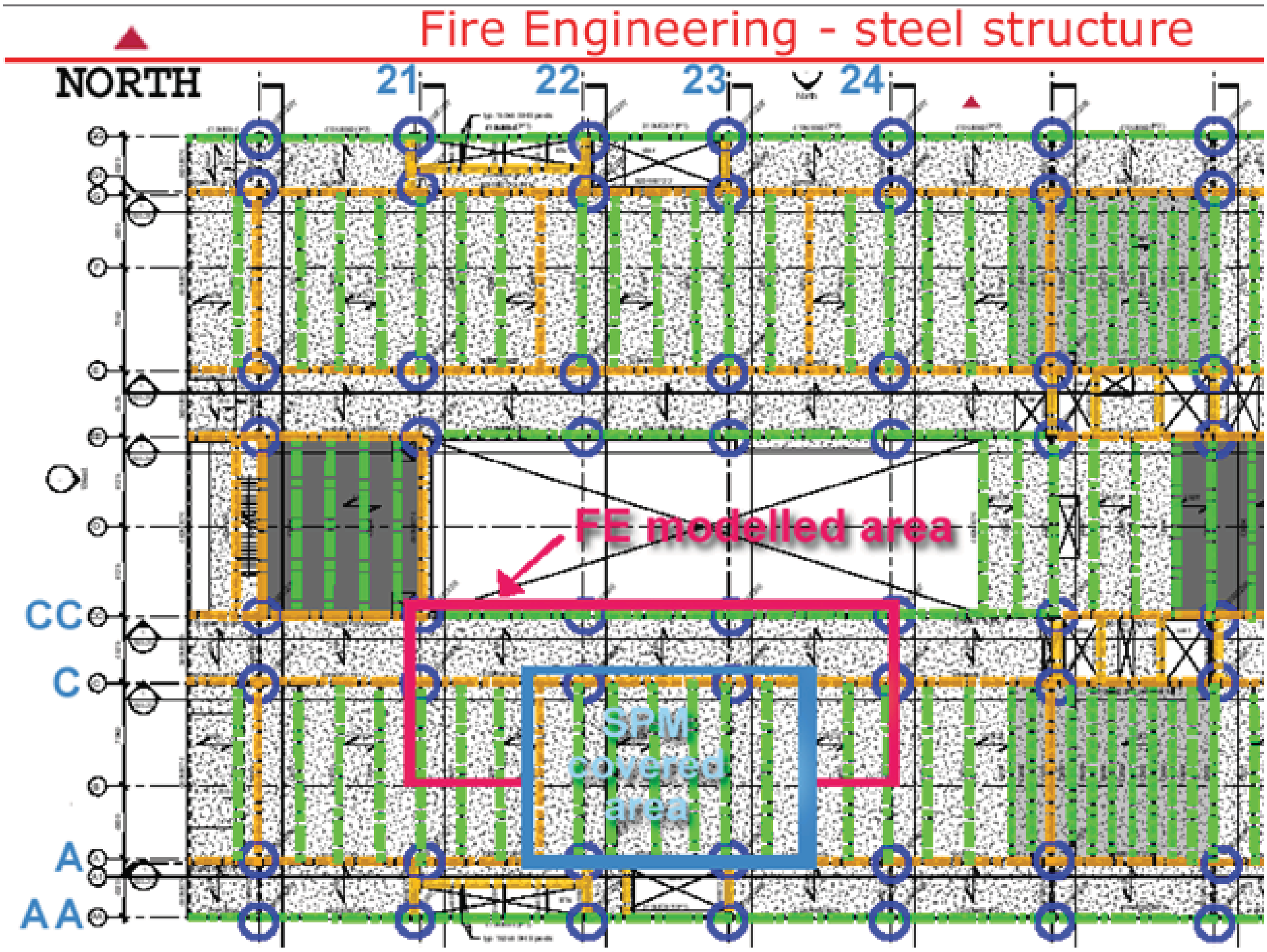

Fig. 5.3 Typical floor showing the fire engineering design and part of the building modelled in the finite element analyses

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.

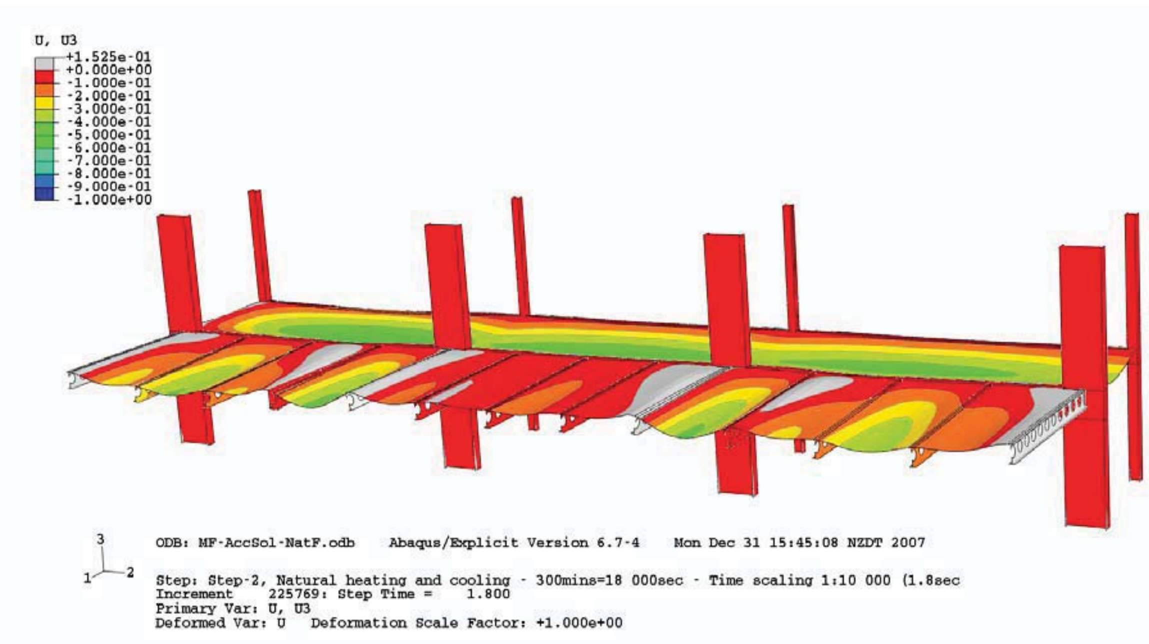

Fig. 5.4 Prescriptive solution with full fire protection based on Compliance Document C/AS1

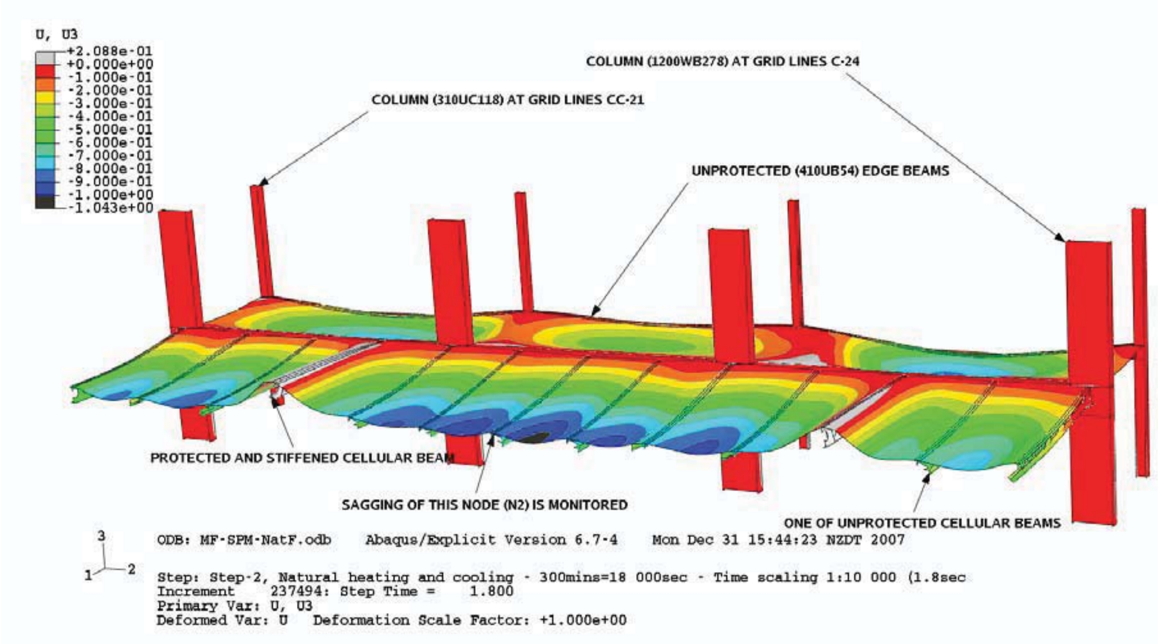

Fig. 5.5 Final design solution based on SPM analysis with partial fire protection

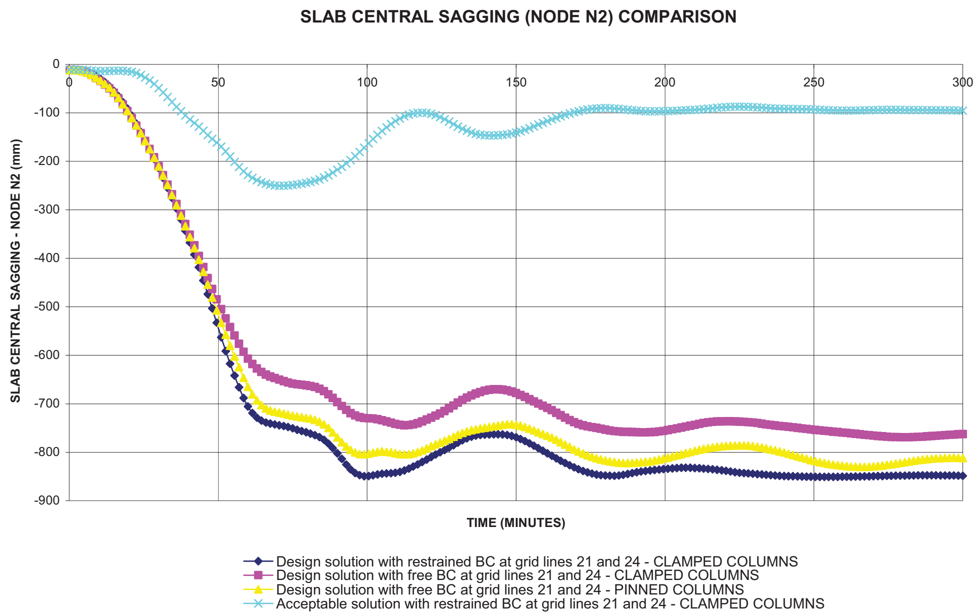

Fig. 5.6 Vertical deflection time history at the centre of slab panel for: partial fire protection (Design solution); and full fire protection (Acceptable Solution) based on based on Compliance Document C/AS1

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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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.

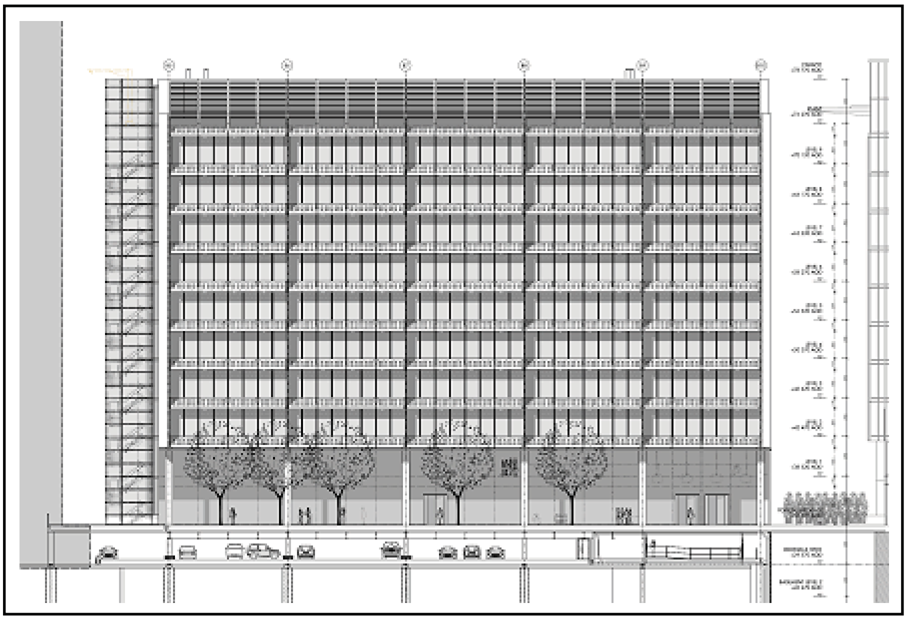

Fig. 6.2 Elevation of the 12-storey 4 Kingdom Street development

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

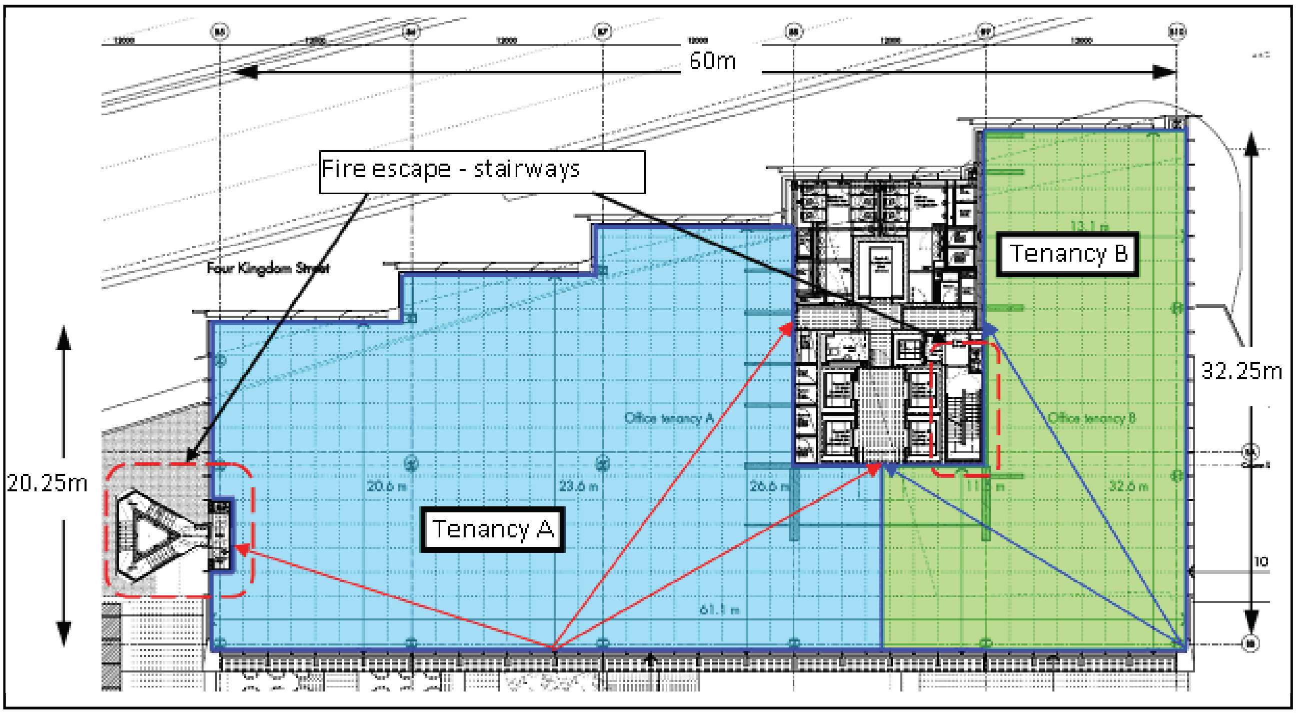

Fig. 6.3 Split-tenancy means of escape

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.

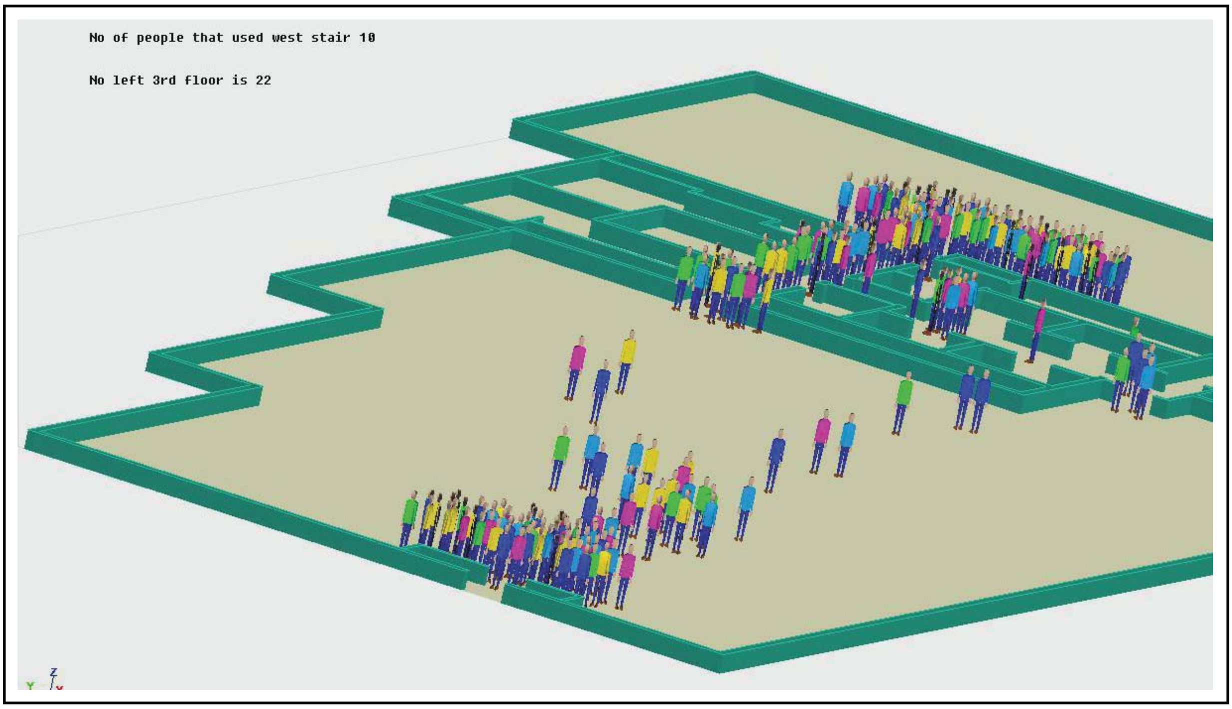

Fig. 6.4 Simulation for occupant evacuation – Typical floor level

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.

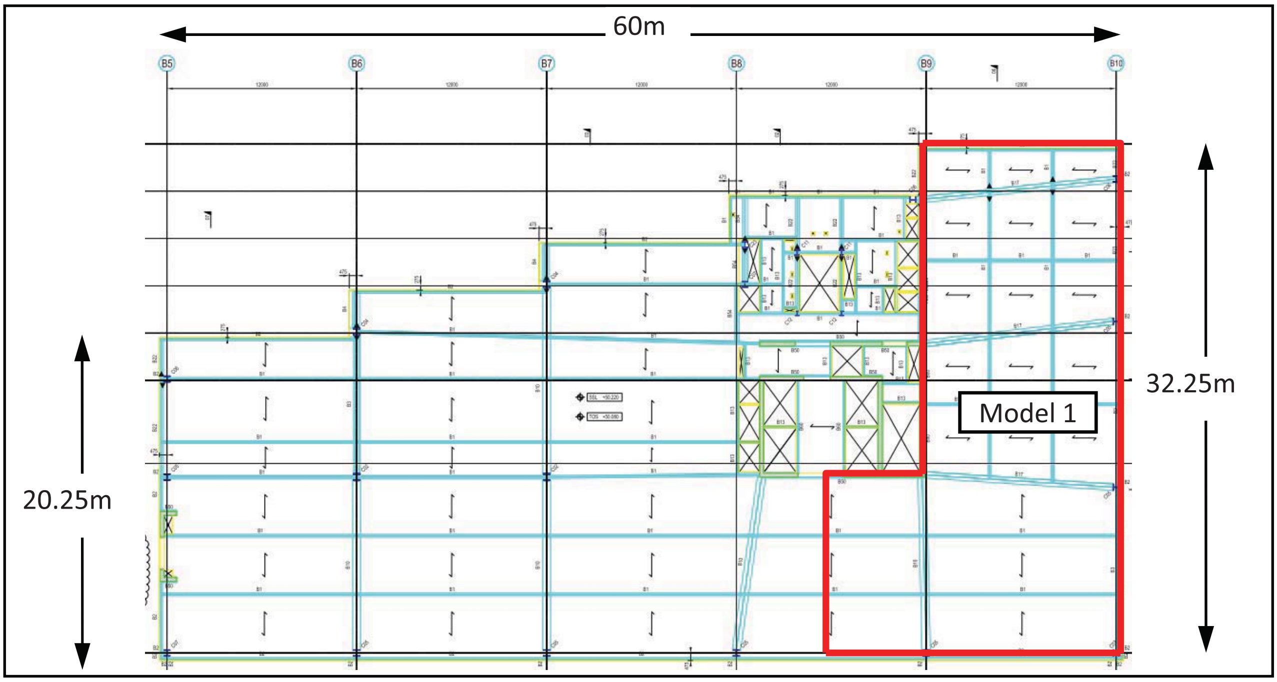

Fig. 6.5 Typical floor plan - Compartmentation Model 1

<u>Structural response – the FE model</u>

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.

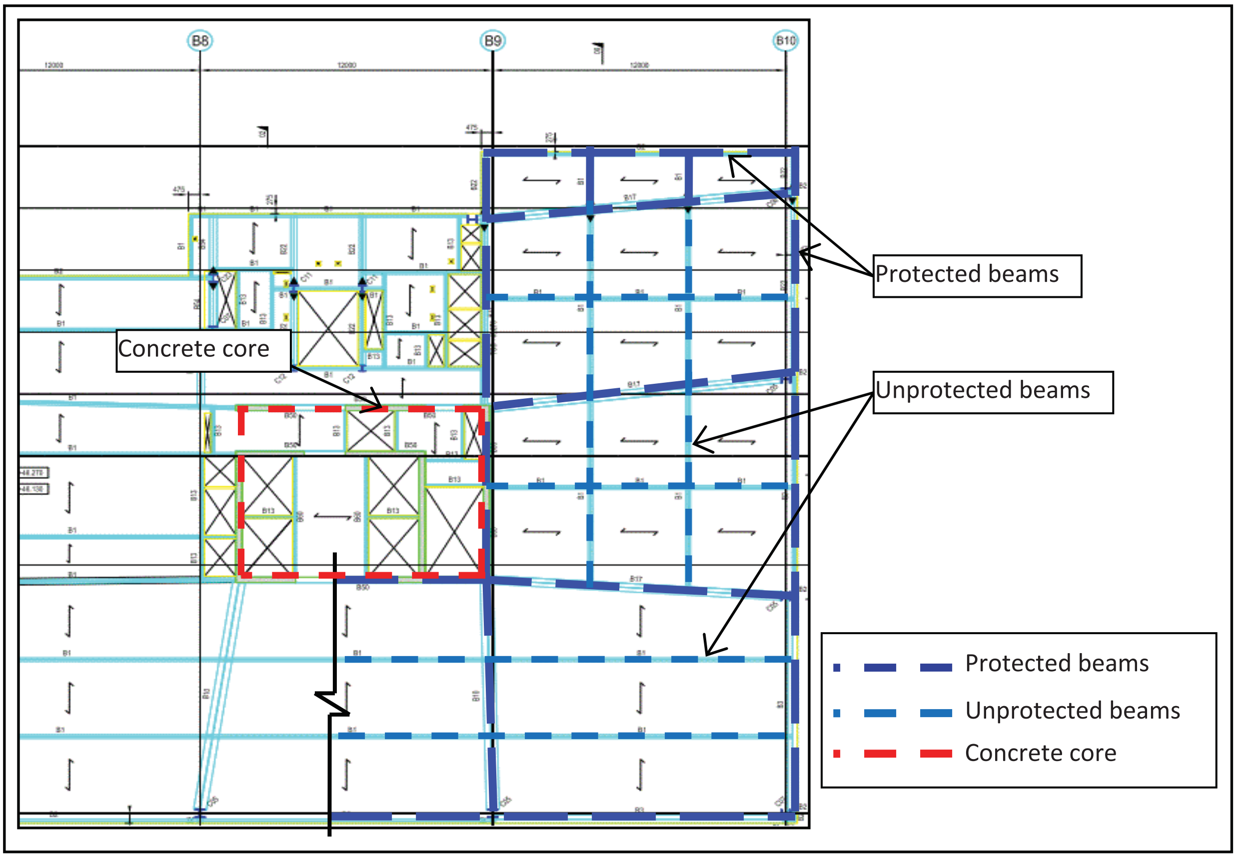

Fig. 6.6 Fire protection scheme of Model 1

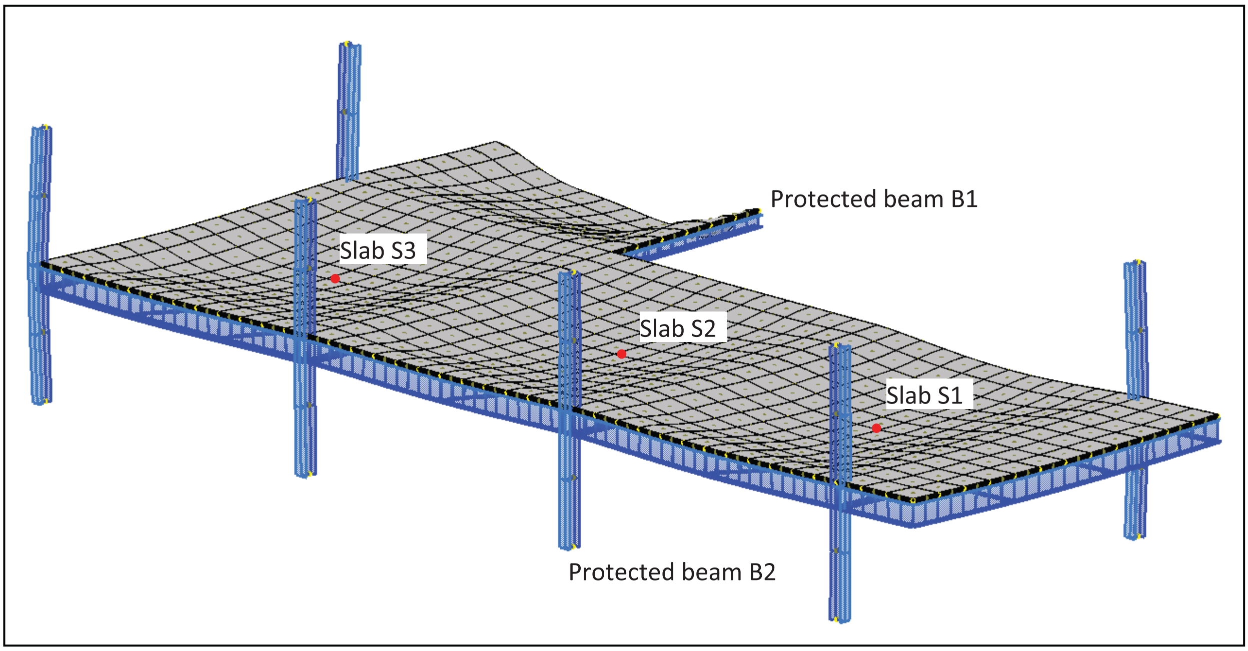

Fig. 6.7 Model 1- predicted global model at elevated temperatures

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.

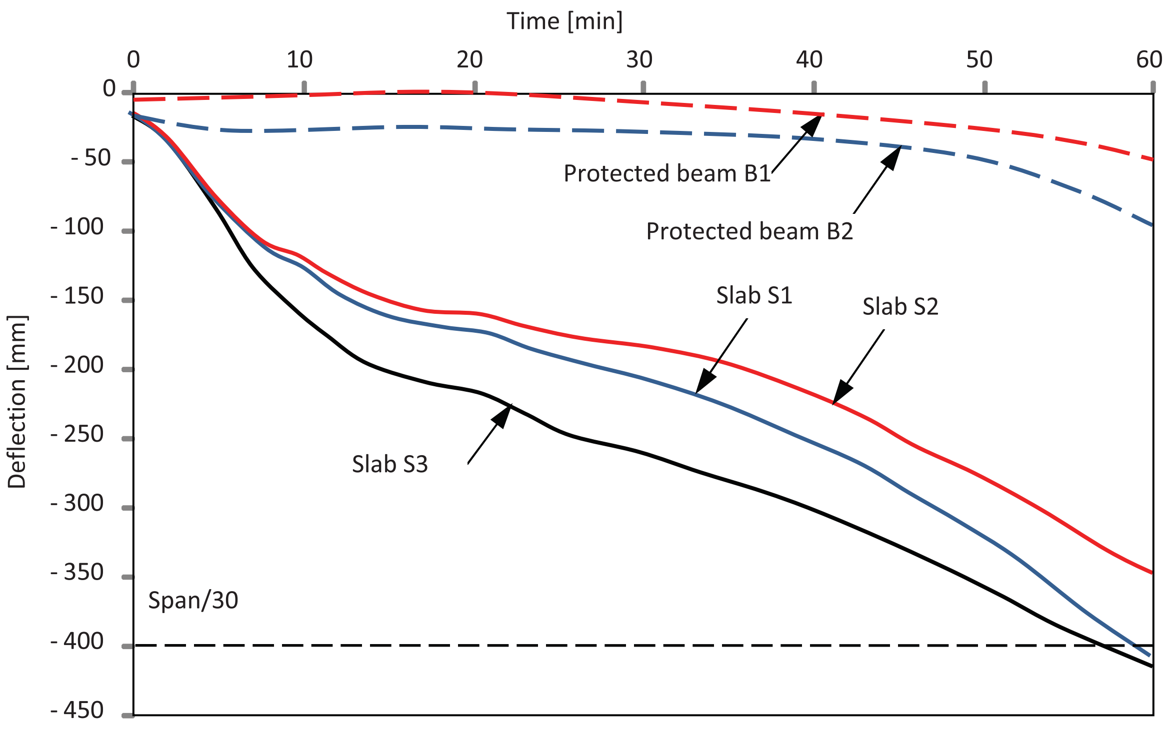

Fig. 6.8 Mid-span deflection

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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