WG1- Emidio Nigro, emidio.nigro@unina.it

WG2- Giuseppe Cefarelli, giuseppe.cefarelli@unina.it

Anna Ferraro, ferraroanna@gmail.com

Gaetano Manfredi, gamanfre@unina.it

Edoardo Cosenza, cosenza@unina.it

23 APPLICATION OF STRUCTURAL FIRE ENGINEERING TO OPEN AND CLOSED CAR PARKS OF C.A.S.E. PROJECT FOR L'AQUILA

Summary

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

FSE requires the choice of a performance level, the definition of design fire scenarios, the choice of fire models and several numerical thermo-mechanical analyses. The information provided by a significant research, performed in Europe for open and closed car parks, are used to apply the FSE to the car parks of the new buildings of the C.A.S.E. Project for L'Aquila, characterized by steel columns supporting the seismically isolated superstructure. The results of the application of the FSE approach will be reported and discussed.

23.1 INTRODUCTION

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

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

Current Italian and European codes allow the use of a performance approach through the concept of Fire Safety Engineering. The temperature distribution within the elements and the mechanical and geometric nonlinear structural response are taken into account in the fire performance approach.

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

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

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

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

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

In Italy, the new Technical Code for Constructions has been published in 2008. For the first time in Italy, the fire action is introduced within the definition of the actions on constructions, as an “exceptional load”. The document defines the performance safety levels of buildings according to the safety objectives required by the Directive 89/106/CEE. The Italian Technical Code for Constructions defines five safety performance levels depending on the importance of the building, which establish the damage level that can be accepted. These rules define the fire structural performance requirements and refer to specific technical codes issued by the Italian Ministry of Interior for all activities under the control of the National Fire Brigades, see (Ministry of Interior 2007a and 2007b). The regulations are basically prescriptive and concern several types of building use. However, the performance based fire design and advanced calculation models may be applied either in the lack of prescriptive rules or in the case of “derogation” with respect to prescriptive rules. The performance based design has to developed according to Decree of the Ministry of the Interior of 09/05/2007, see (Ministry of Interior 2007b), titled “Direttive per l’attuazione all’approccio ingegneristico alla sicurezza antincendio”. The fire design, according to D.M. 09/05/2007, summarized in Fig. 23.1, is divided in two stages: the first is preliminary analysis, i.e. qualitative analysis, while the second is quantitative analysis. Between the first and second stage, the approval of design fire scenarios by Italian Fire Brigade (Vigili del Fuoco) is needed. Finally, it is important to note that in the current Italian code the performance-based approach does not replace the prescriptive one, but both the approaches coexist. The technical solutions imposed by the prescriptive approach remain one of the possible ways that the designer may choose for the structural fire design.

The following describes the application of FSE (namely the structural behaviour in fire situation) to the car parks in the new buildings of the "C.A.S.E. Project for L'Aquila". This Project was developed in L'Aquila (province of Abruzzo, Italy), after the seismic event of 06/04/2009, in response to the housing emergency. The car parks, placed at the ground floor of the buildings, are mainly built with steel columns that support the seismically isolated superstructure. The Italian prescriptive code provides, for car parks, a fire resistance class for the load-bearing criterion of 90 minutes in standard fire exposure (R90). However, for obtaining the fire resistance class R90 the adoption of protective coatings of steel columns is needed, for which a continuous and accurate maintenance is required: in fact, there is a high possibility of accidental damage of the protective coatings in case of impact with the cars. Moreover, the possibility of damage becomes elevated when a series of acts of vandalism takes place, for example if the car parks are easily accessible and not controlled. Because of the uncertainties on the effectiveness of coatings maintenance, in such cases, their use is not recommended.

Therefore, the lack of protective coatings on steel columns and the structural safety during the fire exposure can be evaluated through the application of performance-based approach, which allows to assess, in a more complete and reliable manner, the structural response with reference to the fire scenarios that can realistically occur.

23.2 CASE STUDY: CAR PARKS

232.1 Building description: analysis of the structural characteristics

Each residential building is built on a seismically isolated plate, with dimensions equal to 21×57 m^2 about, capable of supporting a three-storey building with dimensions in plant equal to 12×48 m^2 about, in addition to the stairs. The buildings (superstructures) are different for architectural and constructive elements; the structures are built with wood materials, reinforced concrete or steel. Each isolated plate (with height of 50 cm) is sustained by steel columns (with height of 260 cm) by the isolation system. In this area, below each seismically isolated plate, the parking (Fig. 23.2) for about 34 cars are contained. In order to distribute the actions on the reinforced concrete foundation plate the columns are allocated on a 6 m × 6 m grid. The dimensions in plant of the compartment are equal to 22×58 m^2; in fact the outside walls, when present, are mismatched 50cm with respect to the vertical projection of the edge of the seismically isolated plate.

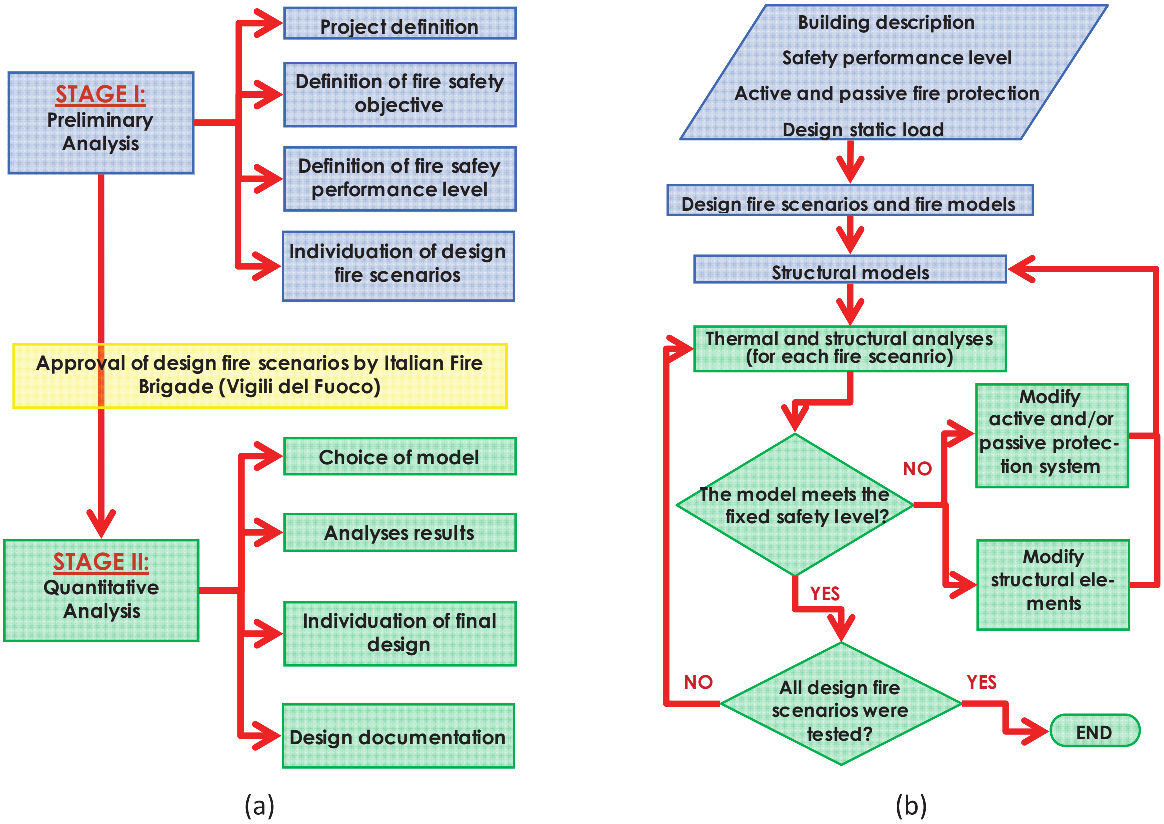


Fig. 23.1 Fire Safety Engineering: Italian code process according to Decree of the Ministry of the Interior of 09/05/2007, see (Ministry of Interior 2007b)

The steel columns are a circular hollow steel section with a capital at the top; this latter is useful a) for transferring, through the isolator unit, the load between the column and the seismically isolated plate and b) as a structure of contrast to the operations of substitution of the isolator unit. The parking area can be fully open on the four sides or partially closed on one or more sides. Therefore, among the various examined cases are present both open car parks and almost completely closed, as well as several intermediate cases.

23.2.2 Choice of safety performance level

In this case study, the objective of fire safety design concerns the mechanical resistance and stability, in fire situation, of the primary structural elements in the zone below the seismically isolated plate. In order to attain this objective, based on the superstructure use (residential buildings), it is sufficient to guarantee that the structures fire resistance requirements for a period consistent with the emergency management are respected (according to performance level III of the Italian Code, see (Ministry of Infrastructure and Transport, 2008)). Nevertheless, in this case a limited damage after the fire exposure has been also required. The damage is quantified in terms of relative vertical displacements between the top of two adjacent columns: in order to limit the finishing damage in the superstructure, the relative vertical displacement must not exceed the limit value, chosen cautiously equals to L/200 (5.0 ‰), where L is the distance between two adjacent columns (L= 6000 mm).

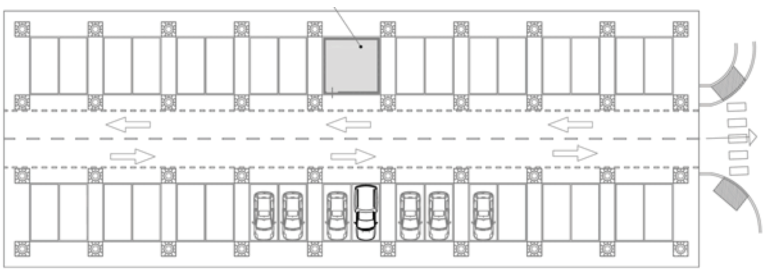


Fig. 23.2 Parking zone

23.2.3 Choice of the active and passive fire protection systems

No specific protection systems (active and/or passive) are provided.

23.2.4 Static and fire design load calculation

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

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

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

Because of the great variability of the superstructure structural type, the fire structural analyses have been carried out, for simplicity and for the benefit of safety, with reference to the maximum combination of exceptional load (maximum axial load on each column equal to 1800 kN).

The design fire load density is closely linked to the type of cars which may be found in the car parks. The cars can be classified according to the thermal energy that can release during the fire. In it is reported the classification of cars (circulating in the period 1995-1998) based on the calorific potential of cars. This classification can be found the final report of CEC agreement 7215-PP/025, concerning a research activity conducted by CTICM (France), Profil-Arbed Recherches (Luxembourg) and TNO (Netherlands) and concluded in 2001. The cars were classified in five categories according to their calorific potential value. In relation to currently circulating cars, it is possible to classify how cars belonging to an inferior or equal category to that of “category 3” (the one having a cylinder capacity not exceeding 2000cc), while those with cylinder capacity upper than 2000cc belong to the “categories 4 and 5” (Tab. 23.1).

The percentage of vehicles, circulating in Abruzzo at the date of 31/12/2008, of cylinder capacity exceeding 2000cc is equal to 6.6% of the total vehicles (from statistics of A.C.I. - Italian Automobile Club). Therefore, because each car park has a maximum capacity of 34 vehicles, the percentage of vehicles with cylinder capacity exceeding 2000cc corresponds to 2 vehicles on 34. Moreover, assuming “category 3” as the category representative of the circulating cars, it is possible to assume the contemporary presence of 32 vehicles of category 3 (calorific value equals to 9500 MJ), and 2 cars of superior category or 2 commercial vehicles; for the scope of the analyses, it refers to commercial vehicles (VAN) with calorific value of 9500 MJ containing 250 kg of highly inflammable material (calorific value of 40 MJ/kg), for a total of 19500 MJ.

Tab. 23.1 Classification of cars, see (CEC Agreement, 2001)

Type	Category 1	Category 2	Category 3	Category 4	Category 5
Peugeot	106	306	406	605	806
Renault	Twingo-Clio	Mégane	Laguna	Safrane	Espace
Citroën	Saxo	ZX	Xantia	XM	Evasion
Ford	Fiesta	Escort	Mondeo	Scorpio	Galaxy
Opel	Corsa	Astra	Vectra	Omega	Frontera
Fiat	Punto	Bravo	Tempra	Croma	Ulysse
Wolkswagen	Polo	Golf	Passat	//	Sharan
Theoretical energy	6000 MJ	7500 MJ	9500 MJ	12000 MJ	

Therefore, once defined the distribution of cars, it is possible to determine the design fire load density. This latter can be evaluated from the characteristic fire load density, defined as sum of thermal energies, which are released by combustion of all combustible materials in a space, per unit area related to the floor area. In this case the specific fire load density is:

$$q_f = \frac{H_{tot}}{A_{tot}} = \frac{32 \cdot 9500\,\mathrm{MJ} + 2 \cdot 19500\,\mathrm{MJ}}{1276\,\mathrm{m}^2} = 268.08\ \mathrm{MJ/m^2} \tag{2}$$

Finally, according to EN1991-1-2, the design fire load density can be evaluated as:

$$q_{f,d} = \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \cdot q_f = 1.4 \cdot 1.0 \cdot 0.9 \cdot 268.08\ \mathrm{MJ/m^2} \cong 340\ \mathrm{MJ/m^2} \tag{3}$$

where δ_{q1}=1.4 (factor taking into account the fire activation risk due to the size of the compartment), δ_{q2}=1.0 (factor taking into account the fire activation risk due to the type of occupancy) and δ_n=0.9 (factor taking into account the different active fire fighting measures i) are defined according to Italian code (Ministry of Interior, 2007a).

23.2.5 Fire design scenarios and Fire model

The fire scenario is significantly affected, among other things, by the geometry and ventilation conditions of the compartment. As regards the evaluation of number of vehicles involved in the fire and the timing of fire initiation by a car to adjacent one, reference is made to the informations from the final report of CEC agreement 7215-PP/025, where are reported the results of real fires in car parks and full scale tests conducted in Vernon (France), both in the presence of free ventilation and with limited ventilation. These results have allowed the drafting of guidelines INERIS currently used in France for the definition of fire scenarios in car parks according to Decree of French Ministry of Interior of 9 may 2006, see (Arrête, 2006).

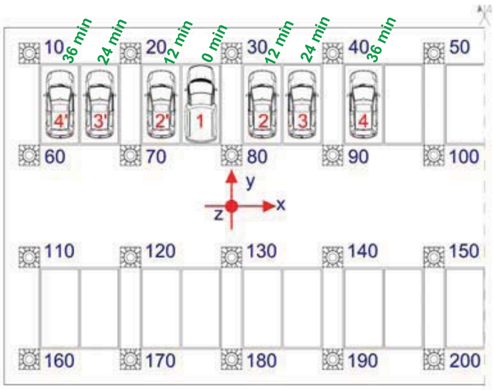
It is necessary to distinguish the car parks open on all their sides by those partially open (openings limited or absent on one or more sides). The presence of natural ventilation in open car parks does not allow the achievement of the flashover conditions: for this reason the phenomenon remains for the entire fire duration of “pre-flashover” type. In these conditions a limited number of vehicles, near the ignition source, burn. In partially open car parks, instead, it is possible that the fire involved all of the cars.

Therefore, the identification of the more dangerous fire scenarios for the structural stability is to define the position and the number of cars that may be involved in the fire and cause the more dangerous thermal action, between those realistically conceivable, for the supporting structure building.

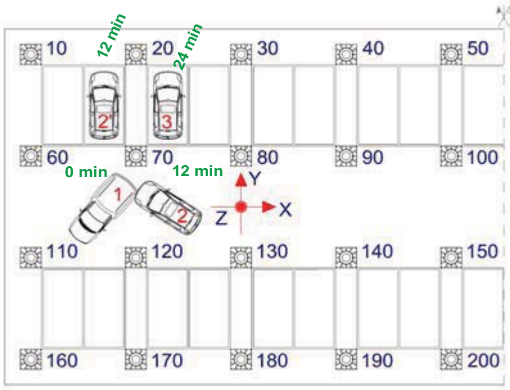
By applying the criteria proposed in the aforementioned guidelines to *car parks open on all sides* the types of distribution of the cars described in Tab. 23.2 are chosen, with a fire propagation time from car to adjacent one equals to 12 min. Thanks to the symmetry of car parks’ structures, in order to maximize the fire effects, the vehicles are located according to the Fig. 23.3.

Tab. 23.2 Cars distribution for localised fire scenarios (pre-flashover)

Type 1 (L1)	7 vehicles, of which 1 central VAN and 6 cars, that burn with a fire propagation time from car to adjacent one equals to 12 min from the VAN.
Type 2 (L2)	4 vehicles, of which 1 central VAN and 3 cars surrounding a column, that burn with a fire propagation time from car to adjacent one equals to 12 min from the VAN.



Scenario L1



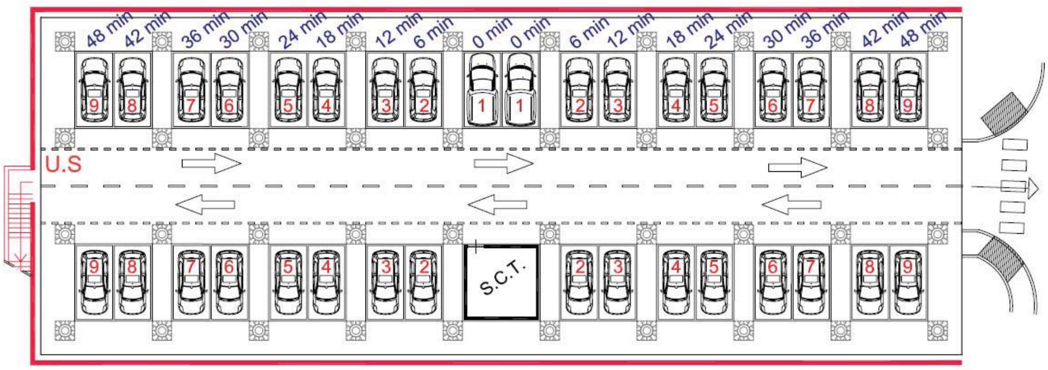
Scenario L2

Fig. 23.3 Pre-flashover fire scenarios

Instead, with regard to the *partially open car parks* (openings limited or absent on one or more sides), in addition to considering the localised fire scenarios (pre-flashover), there must be considered generalized fire scenarios as well (post-flashover), which involve, in the extreme event that the whole of car space available is occupied, all present vehicles. The time of the spread chosen for this case from a car to adjacent one is 6 min, in agreement with the results of the above experimental full-scale tests with limited ventilation. Therefore, the types of distribution of the cars (with 6% of VAN) described in Tab. 23.3 are chosen, with a fire propagation time from a car to adjacent one equals to 6 min. The vehicles location and the spread time are reported in the Fig. 23.4.

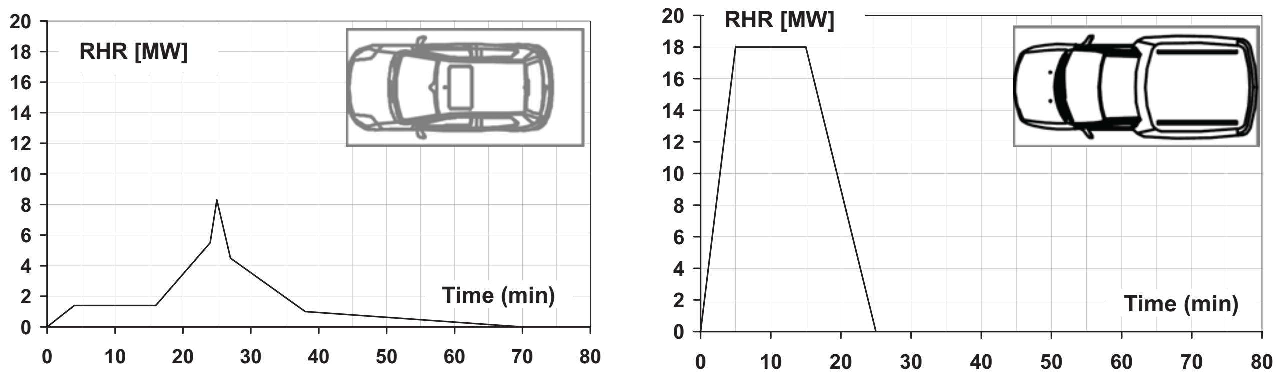
Tab. 23.3 Cars distribution for generalized fire scenarios (post-flashover)

Scenario D1 (6% VAN)	34 vehicles, of which 2 central VAN and 32 cars, that burn with a fire propagation time from car to adjacent one equals to 6 min from the VAN



Scenario D1 (6% VAN)
Fig. 23.4 Post-flashover fire scenario

For localised fire (pre-flashover) the temperature in the fire flame and plume and the surrounding gas are not uniform, and need to be determined separately. Instead in a post-flashover fire the temperature is assumed to be uniform within the fire compartment. Eurocode 1 Part 1-2 (Annex C, EN1991-1-2) provides a simple calculation method (Hasemi's Method) for determining the thermal action of localised fires of compartments in which the input data is the heat released by combustible products (in this case the single car) as a function of time (namely Rate of Heat Release - RHR). The Rate of Heat Release curves for the single burning car of "category 3" and for a single burning VAN are provided by calorimetric hood tests reported in the final report of the quoted CEC agreement 7215-PP/025. In Fig. 23.5 are reported the RHR curve of a) car of category 3 and b) VAN obtained by fitting the experimental results.



a) Category 3 (car) b) VAN

Fig. 23.5 RHR curves

For the generalised fire scenarios, the temperatures in the compartment are evaluated by the software OZone ver. 2.2 (Cadorin & Franssen, 2003). OZone ver. 2.2 is a zone modelling software (according to the Appendix D of EN 1991-1-2) which the temperature development of the gases within a compartment during the course of a fire allows to evaluate from the input data as: the geometric characteristics of the compartment, the thermal characteristics of the materials of which it consists, the ventilation conditions and the rate of heat release obtained through the overlapping time in the sequence of initiation of the single car RHR curves.

The results of the two fire models (localised fire and generalized fire) is different: in fact, the Hasemi's method gives the heat flux received by the fire exposed surfaces at the level of the ceiling, while the zone model provides the temperature within the compartment.

For the localised scenarios (L1 and L2), in Fig. 23.6 the heat flux received by some significant steel columns are reported.

For the generalized fire scenarios, in Fig. 23.7 are reported the compartment time-temperature curves given from the zone model of the fire scenarios D1. Because of the several possible ventilation conditions for the car parks case study, 7 different ventilations classes (V1,...,V7) were considered. In order to maximize the fire exposure, the structural analyses have been referred only to the ventilation condition V1; in fact, because of the fact that the temperature of the structural elements is mainly dependent on the growth phase and on the maximum fire temperature, the fire with ventilation conditions V1 is the most dangerous one.

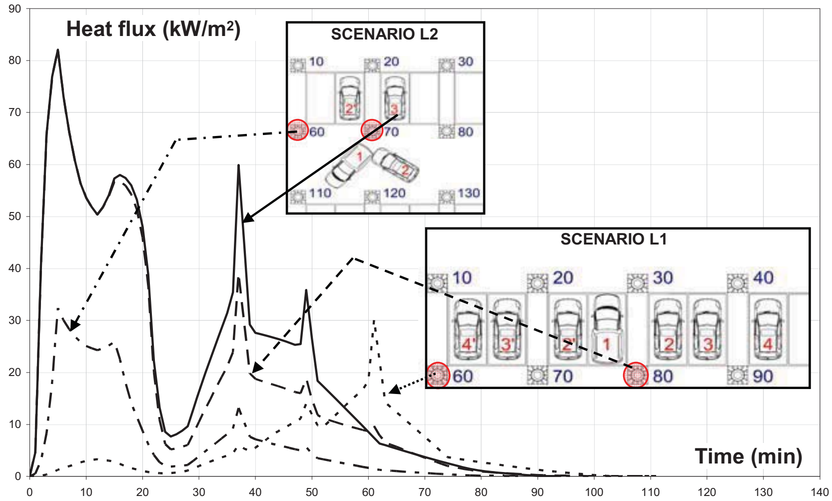


Fig. 23.6 Thermal flux from Hasemi's method

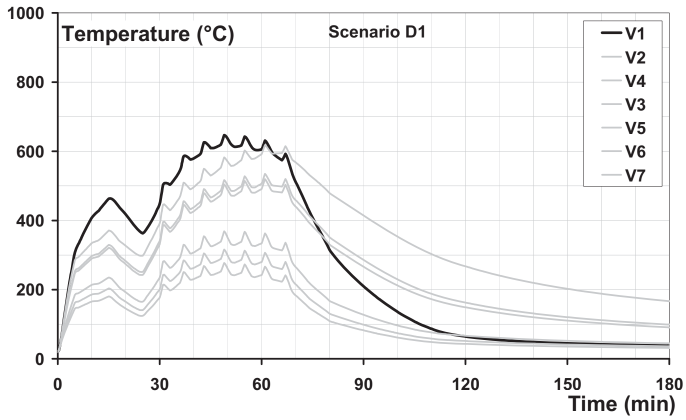
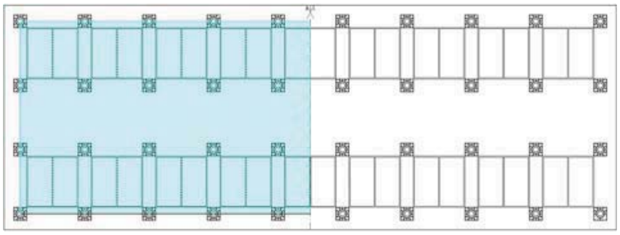


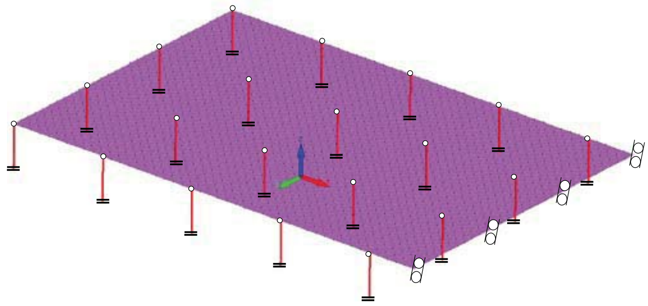
Fig. 23.7 Time-Temperature curves for post-flashover fire

23.2.6 Structural model and fire safety assessment

In order to limit the analysis time without compromising the accuracy of the results, the thermo-mechanical analyses, for each fire scenario, have been conducted with the reference to the substructure highlighted in Fig. 23.8 (Nigro, 2010). The substructure extension allows assessing in an appropriate way both the thermal field and the hyperstatic effects induced by different thermal expansions of steel columns and bending of the concrete reinforced slab. Along the edge a constraint is introduced for the horizontal movements in the longitudinal direction and for the rotations around transverse axis. This constraint condition, thanks to the structural symmetry, is fully congruent for the analysis in normal temperature conditions and for the generalised fire scenarios (scenario D1), while it is on the safe side for the other scenarios (localised fire scenarios), maximizing, thanks to the infinite rotational stiffness, the hyperstatic effects induced on the columns by slab thermal curvature. The steel columns are fixed at the base and linked to the superstructure slab with a hinge.



Substructures



Static scheme

Fig. 23.8 Thermo-mechanical model of the structure

For each fire scenario, the global thermal-mechanical structural analyses of the substructure in Fig. 23.8 are conducted by using the non linear software SAFIR2007a (Franssen, 2005), developed at the University of Liege (Belgium), which performs the structural analysis under fire conditions. The steel columns are modelled with beam elements with circular cross-section, while the reinforced concrete slab is modelled with shell elements. In addition to the global analysis, for each fire scenario, in order to calculate more accurately the thermal field and stresses distribution in the capitals above the columns and to assess the possible local buckling, a detailed thermo-mechanical analyses has been conducted with reference to the more stressed and heated column. The 3D modelling (Fig. 23.9) have been developed with the finite element software ABAQUS/standard. The thermal exposure conditions were considered according to Fig. 23.6 and Fig. 23.7. The axial load corresponds to the axial load obtained by the global structural analyses.

Fig. 23.9 3D thermo-mechanical model of the column

23.2.7 Analyses results

For sake of brevity, the results of structural response in fire situation are reported only with reference to the fire scenario L2, which appears more unfavourable (Fig. 23.10). The maximum temperatures reached in the columns do not exceed 600°C (Fig. 23.10-a). It appears clear that the highest temperatures reached in columns, namely those nearest to the VAN (characterized by a higher calorific potential), reach a maximum temperature of about 580°C in correspondence of a fire exposure time of about 20 minutes. The thermal action produces both in the columns and slabs several thermal expansions. Because of the thermal curvature of the slab the columns axial load increases (Fig. 23.10-b). The axial load is further amplified from the differential thermal elongation (Fig. 23.10-c) of columns, exposed to different thermal conditions, which is constrained from slab shear stiffness.

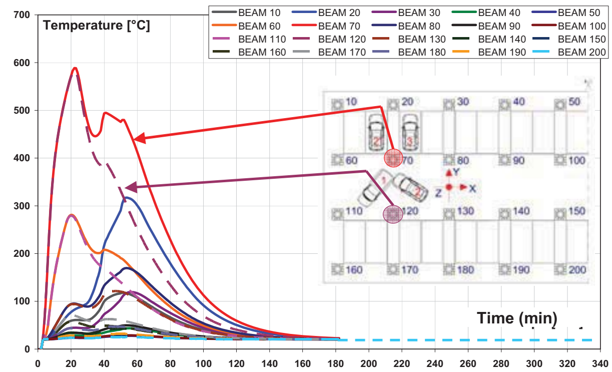
The thermal expansion induced by fire leads to the columns elongation with consequential upwards displacement (Fig. 23.10-c). The displacement reflects, in general, the temperatures trend. However, the reduction of stiffness that the structural elements suffer, if constrained to high temperatures, may lead to a premature reversal development in displacement. In fact, to the column 70 of the scenario L2, the displacements increase until a fire exposure time of about 18 minutes (corresponding to steel column temperature of about 570°C), reversing later on their trends because of the presence of a high axial load and the elastic modulus reduction with the temperature. The shortening is subsequently determined also by the reduction of the temperature which begins after about 20 minutes of fire exposure. By following the almost complete cooling phase of the columns it is possible to notice a small residual deformation; in fact, the final displacement, after the fire exposure, is slightly different from the initial elastic one (namely, before the start of fire).

Moreover, the maximum differential displacement reached between the adjacent columns is very small. In fact, the maximum differential displacement, during the fire exposure, between the column 120 and column 130 is of about 16mm at the time of exposure of about 20 minutes. This value corresponds to 2.6 ‰ (16mm/6000mm), definitely below the limit value of 5.0 ‰ taken as the acceptance criteria.

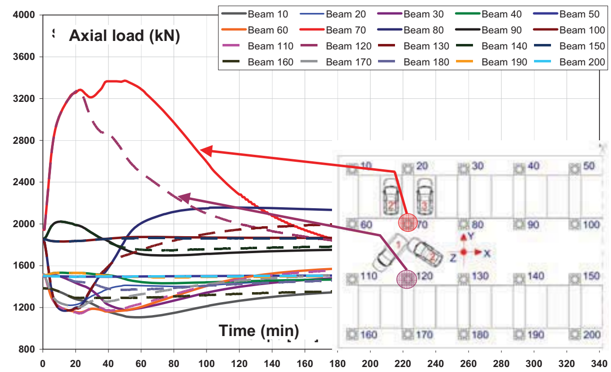
The checks on resistance automatically carried out by the software are also integrated by comparing the axial load during the fire to the axial load resistance of the columns evaluated according to EN1993-1-2. In Fig. 23.10-d is showed that the more loaded column, even when the maximum temperature is reached, still has a significant reserve of resistance.

As regards the detailed analysis of the column, the displacement at the head of column is very similar to those obtained in the global structural analyses (Fig. 23.11). The final displacement is about 5mm in the central area of capital and about 2mm in the tube head: this is due to the plastic strain which has developed in the tube and in the capital (mainly in the zone of load application) during the fire exposure.

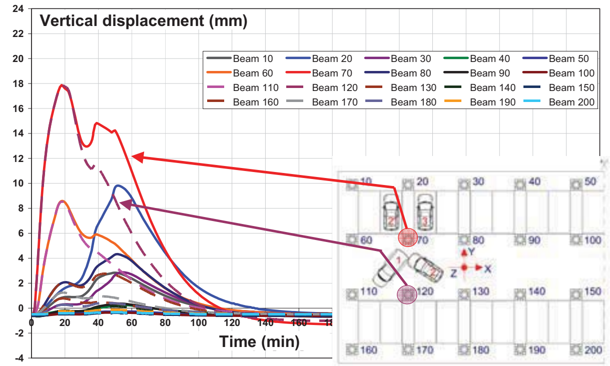
Similar considerations are also valid for the other fire scenarios. Therefore it can be concluded that the structure, and in particular the columns in the absence of any protection system against fire, during the design fire exposure perform an adequate load-bearing capacity, including the cooling phase.



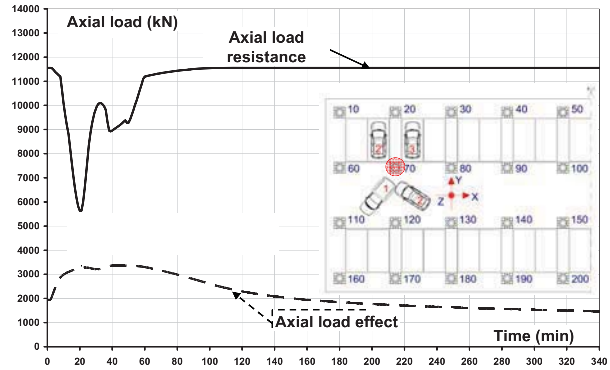
a) Time-temperature curves for the columns



b) Time-axial load curves for the columns

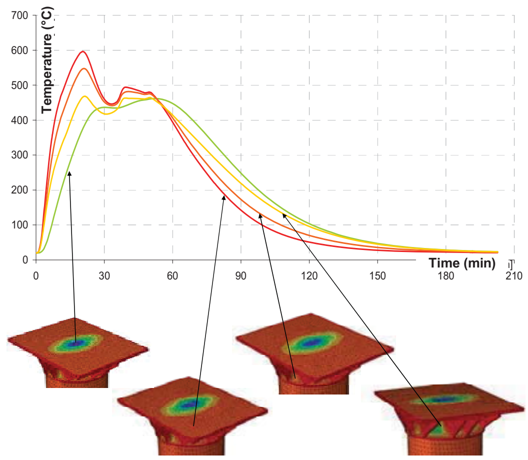


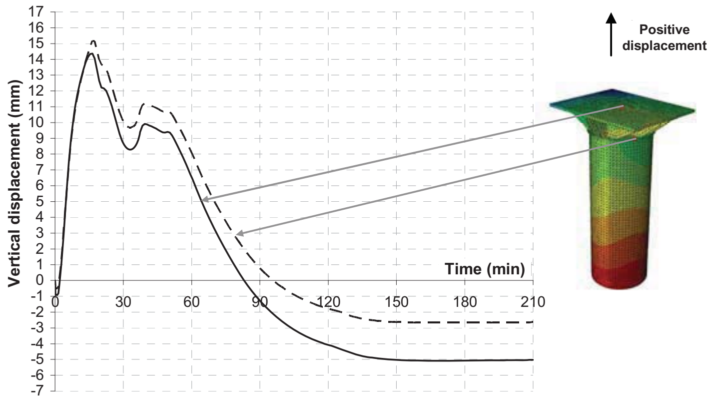
c) Time-displacement curves for the columns' top



d) Stability check according to EN1993-1-2 (column 70)

Fig. 23.10 Scenario L2 – Global analyses





a) Columns time-temperature

b) Columns head time-displacement

Fig. 23.11 Scenario L2 – Detailed analysis.

23.3 SUMMARY

The Fire Safety Engineering approach thanks to advanced calculation models both for fire and for thermo-mechanical analysis of the structure, allows simulating the response behaviour of the structure exposed to “natural” fire scenarios. The FSE application to the car parks, to which this paper is dedicated, is allowed thanks to the information about the possible fire scenarios provided by the European Research Project CEC agreement 7215-PP/025 (2001). These fire scenarios may be of localised or generalised type as a function of the geometry and openings of compartment, namely of the ventilation conditions.

A natural fire is characterized by a heating phase and by a cooling phase. The thermal gradient in structural elements produced by the cooling phase is opposite to that produced by the heating phase. During the heating fire exposure there is a non-linear structural behaviour and plastic strains can be achieved in the structural members; for this reason, during the cooling phase the structure is different from the initial one. Therefore, after the cooling phase the stresses and the forces in the structural element can be different from the ones that could be found before the fire exposure.

The stresses and forces induced by constrained thermal deformations may cause structural collapse; however, they cannot be fully controlled by the prescriptive approach, as this approach is based on the assumption of a standard fire curve which increases unrealistically.

Finally, the thermo-mechanical analyses in fire situations for the described case study, consisting of the car parks located at the ground floor of buildings of the C.A.S.E. Project - L’Aquila, showed that the structures, and in particular the steel columns, considered unprotected, satisfy the performance level set to the design fire scenarios, also thanks to an overstrength in normal condition design.

References

ABAQUS Standard/explicit User’s Manual (2008), Hibbit Karlsson and Soresen, Inc. Vol 1-2-3, Version 6.7, USA 2008.

Arrête, 2006: Règlement de sécurité contre les risques d’incendie et de panique dans les parcs de stationnement couverts, Ministère de l’Intérieur et de l’Aménagement du Territoire 9 Mai 2006.

Cadorin, 2003: Cadorin J-F., Franssen J-M., A tool to design steel elements submitted to compartment fires-Ozone V2. Part 1: pre and post flashover compartment fire model, Fire Safety Journal, Vol 38 No. 5, pp 395-427.

CEC Agreement 7215 - PP/025 (2001), Demonstration of Real Fire Tests in Car Parks and High Buildings.

Construction Product Directive (1988), 89/106/CEE, Construction of European Community, December.

EN 1991-1-2 (2002), Eurocode 1. Actions on structures - Part 1-2: General actions - actions on structures exposed to fire, November.

EN 1992-1-2 2004. Eurocode 2. Design of concrete structures – Part 1-2: General Rules – Structural Fire Design. March.

EN 1993-1-2, Eurocode 3. Design of steel structures - Part 1-2: General rules - Structural fire design, April 2005.

Franssen J.-M., 2005: Franssen J.-M., SAFIR. A Thermal/Structural Program Modelling Structures under Fire. Engineering Journal, A.I.S.C., Vol 42, No. 3 (2005), 143-158.

INERIS, 2001: INERIS, Parcs de stationnement en superstructure largement ventiles. Avis d'expert sur les scénarios d'incendie, Ottobre.

ISO/TR 13387 (1999), Fire safety engineering.

Ministry of Interior (Italian Government) 2007a. Decree 09/03/2007, Prestazioni di resistenza al fuoco delle costruzioni nelle attività soggette al controllo del Corpo nazionale dei vigili del fuoco. GU n. 74 of 29/03/2007.

Ministry of Interior (Italian Government) 2007b. Decree 09/05/2007, Direttive per l'attuazione dell'approccio ingegneristico alla sicurezza antincendio. GU n. 117 of 22/05/2007.

Ministry of Infrastructure and Transport (Italian Government) 2008. Technical Code for the Constructions. G.U. n. 29 of 14/02/2008.

Nigro, 2010: Nigro E., Ferraro A., Cefarelli G., Application of FSE approach to the structural fire safety assessment of steel-concrete composite structures, Proceedings of COST Action C26 Conference, 16-18 September 2010, Naples, Italy.

Nigro, 2011a: Nigro E., Cefarelli G., Ferraro A., Cosenza E. and Manfredi G., Fire Safety Engineering for open and closed car parks: C.A.S.E. Project for L'Aquila, in: Applied Mechanics and Materials, Vol. 82 (2011), Performance, Protection and Strengthening of Structures under Extreme Loading, Trans Tech Publications, ISBN-13 978-3-03785-217-0.

Nigro, 2011b: Nigro E., Cefarelli G., Ferraro A., Cosenza E. and Manfredi G., Fire Safety Engineering applied to car parks of C.A.S.E. project for L'Aquila buildings: General topics (in italian), in: Proceedings of XXIII Congresso CTA Le giornate Italiane della costruzione in acciaio, Ischia (NA), 09-12 ottobre 2011.

Nigro, 2011c: Nigro E., Cefarelli G., Ferraro A., Sannino D., Cosenza E. and Manfredi G., Fire Safety Engineering applied to car parks of C.A.S.E. project for L'Aquila buildings: Detail topics (in italian), in: Proceedings of XXIII Congresso CTA Le giornate Italiane della costruzione in acciaio, Ischia (NA), 09-12 ottobre 2011.

WG2 - Cécile Haremza, haremza@dec.uc.pt
WG2 - Aldina Santiago, aldina@dec.uc.pt
WG1 - João Paulo Rodrigues, jpaulocr@dec.uc.pt
WG3 - Fernanda Lopes, fernanda@dec.uc.pt

24 FIRE SAFETY DESIGN OF A MODULAR STEEL SYSTEM FOR HOTEL ROOMS

Summary

This paper presents a constructional solution developed for a modular steel structure, taking into account the fire safety design. This constructional solutionis intended to be used for construction of hotelrooms. According to the Portuguese Regulation, the minimum standard fire resistance of structural beams and columns is REI90, for the slab is REI 60 and for the partition elements is EI60 (Portaria nº1532/2008). Hot-rolled steel profiles are considered in the structural elements, and gypsum boards panels are used for the partition elements. The proposals for the partition elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data from manufactures of fire protection materials. Eurocodes 1 and 3 Part 1.2 (NP EN 1991-1-2:2010 and EN 1993-1-2:2005) are used for the design of the structural elements, taking into account the standard ISO 834fire curve.

24.1 GENERAL BUILDING DESCRIPTION

The study is based on a 4-floor hotel which isalready built using traditional construction in Vilamoura, Portugal. According to the Portuguese Regulation,this building is classified as type VII, 3rd category: Hotel and Catering lower than 28 m of high (DL nº220/2008). The unusual feature studied in this paper is the constructive solution for hotel buildings with prefabricated modules. The modules are made up with a steel structure (hot-rolled steel profiles) and partition elements. The evaluation of the fire safety performance of the structural solutions,developed (or proposed) by *OPWAY NovasTecnologias*, is the main topic of this study. Only one structural solution for the modular construction is detailed in this paper, taking into account the fire safety design. Two situations are studied: case A,the prefabricated module, for which columns and beams are respectively inserted in to walls and drop ceiling; case B, two modules are joined into a single one to make, for example, spacious rooms, and columns and beams are isolated. The solutions studied are summarized in Tab. 24.1. The proposals for the partition elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data of fire protection materials from manufactures. Tab. 24.2 shows the materials and their propertiesconsidered in this study.

Tab. 24.1 Summary of the studies

	Regulatory requirements	**Case study**
Walls	EI60	
Drop ceilings	EI60	
Columns	REI90	Case A: into walls (HEB 140)
		Case B: isolated columns (HEB 140 + Promatec-H20 mm thick)
Beams	REI90	Case A: into drop ceilings (HEA 140 + HEB 140)
		Case B: isolated beam (HEA 140 + Knauf Fireboard 35 mm thick)
Slabs	REI60	

Tab. 24.2 Properties of the recommended protection materials

	Plasterboard type *Knauf DF*	Plasterboard type *Knauf standard*	Plasterboard type *Knauf Fireboard*	Rock-wool	Calcium silicate board type *Promatec-H*
l_p (W/mK)	0.25	0.25	0.25	0.04*	0.175
c_p (J/KgK)	1700*	1700*	1200*	840*	1200*
r_p (kg/m^3)	750*	750*	800	40	870
p (%)	20*	20*	20*	≈ 0*	≈ 0*
e (mm)	12.5	12.5	35	50/75	20

* Values tabulated in general bibliography; manufacturer's data do not provide this information.

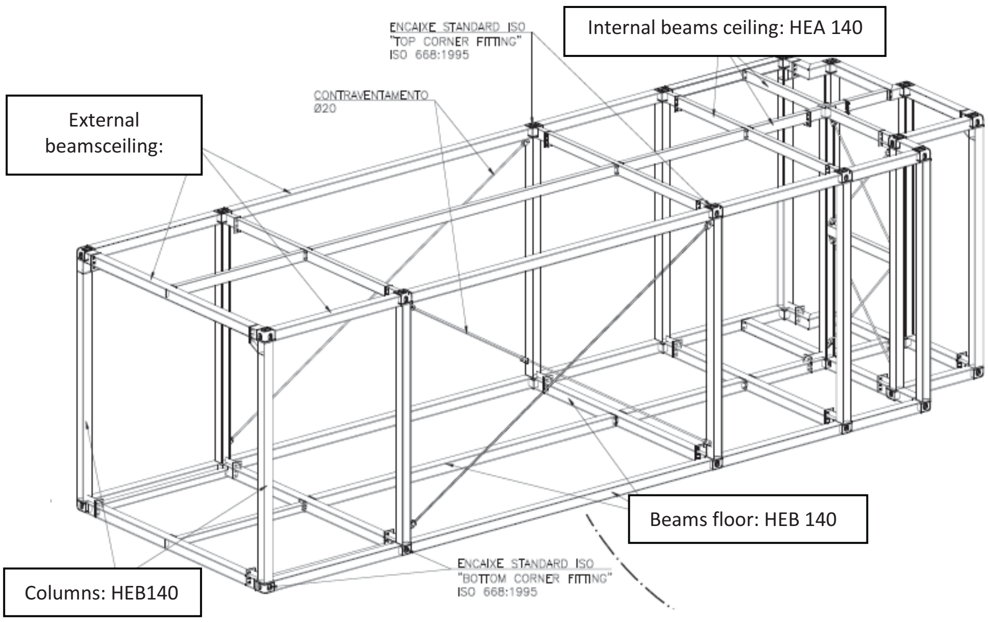


Fig. 24.1 Pre-fabricated module

24.2 ON WHAT PART OF THE PROJECT IS FIRE ENGINEERING USED AND THE PURPOSE OF CHOOSING A FIRE ENGINEERED APPROACH

Prefabricated modules are intended to be used for construction of hotel rooms, and the fire engineered approach is chosen in order to check the fire safety design of the modules.The design of structural elements (beams, columns and slabs) is performed according to Eurocodes 1 and 3, Part 1-2 (NP EN 1991-1-2:2010 and EN 1993-1-2:2005). The calculation procedure is based on a prescriptive approach and is carried out in the field of temperature.In many cases, the geometrical definition of the structural profiles insulationdid not fit to the typologies indicated in Eurocodes. Taking into account the safety of the structure, simplifications and approximations are performed to fit the problem in the Eurocodes typologies and formulations.

24.3 REGULATORY REQUIREMENTS FOR THE FIRE ENGINEERED PART OF THE PROJECT

24.3.1 Walls, ceilings and slabs

According to the Portuguese Regulation (Portarianº1532/2008), the building considered in this case study is part of thefollowing hazardous locations:

D - Establishment intended to receive children less than six years old or people with limited mobility or without ability of perception and response to an alarm (DL nº 220/2008, article 10);

E- Establishmentintended for overnight stays (DL nº 220/2008, article 10).

The minimum fire resistance of elements under ISO 834 standard fire curve for hazardous location D and E are given in articles 22 and 23 of Portarianº1532/2008 (Tab. 24.3). The most unfavourable situation for this study caseis toconsider thehazardous locationD.

Tab. 24.3 Hazardous locations for the analysed building

Hazardous locations E	
Construction elements	**Minimum standard fire resistance**
Non-resistant walls	EI30
Floors and resistant walls	REI30
Doors	E15C
Hazardous locationD	
Construction elements	**Minimum standard fire resistance**
Non-resistant walls	EI60
Floors and resistant walls	REI60
Doors	E30C

Regarding the overall fire compartmentation, in general, the various levels must be defined as differentfire compartments. The maximum area of a fire compartment by floor for a type of use VII should be 1600m^2. These compartments must be isolated by building elements with a resistance class EI60 or REI60for type VIIuse, providing, at the minimum,spans with standard fire resistance class E30 (Portarianº 1532/2008, article 18).

24.3.2 Structural profiles (beams and columns)

According to the Portaria nº1532/2008, article 15, the minimum standard fire resistance of structural elements is given in function of the building category for different types of use. The present case study refers to a *type of use VII – accommodation and food services* (DL nº 220/2008, article 8) that corresponds to *buildings or parts of buildings receiving people, providing temporary accommodation or performing food services and drink activities, in exclusive occupation or not, namely those for tourism enterprises, local accommodation, food servicesor drinksestablishments, dormitories and, when not inserted in school establishment, student residences and holiday camps, campsites and caravanningbeing excluded and considered as spaces of use-typeIX*. The risk categories usedforuse-typeVII are presented in Tab. 24.4 (corresponds to Table VI of Annex III of DL nº 220/2008).

Tab. 24.4 Building categories for Use-type VII (UT)

Categories	Reference criteria			Hazardous location E with independent exits directly abroad in the reference plane	Study case (4-floors hotel)
	UT height	Number of persons of the UT			
		Number of persons	Number of personsin hazardous location E		
1st	≤ 9m	≤ 100	≤ 50	Applicable to all	R30; REI30
2nd	≤ 9m	≤ 500	≤ 200	Not applicable	R60; REI60
3rd	**≤ 28m**	**≤ 1500**	**≤ 800**	**Not applicable**	**R90; REI90**
4th	> 28m	> 1500	> 800	Not applicable	R120; REI120

24.4 ASSESSMENT STRATEGY: CONSTRUCTIONAL SOLUTIONS FOR THE PARTITION ELEMENTS

24.4.1 Interior and exterior walls

According to existing experience in this field, it is suggested that, to obtain a fire resistance of EI60, eachwall module should consist of two defined vertical elements(from outside to inside): i) 2 gypsum board panels (theexternal should be type *KnaufDF*and the internal type*Knauf standard*), with 12.5 mm thick each; ii)50 mm of low density rock-woolof 40 kg/m^3(except in the zone of the columns). In the zone of thecolumns,between the inner plate (type *Knauf standard)* and the columnsteel section, it should be placeda calcium silicate board, type*Promatec-H,*with 20 mm thick and 540 mm wide (corresponding tothe column width, with 200 mm moreat each side) as shown in Fig. 24.2. Thechange of the wallinsulation in the columns zone is related to the required fire resistance for columns (REI90), as explained in 24.6.2.

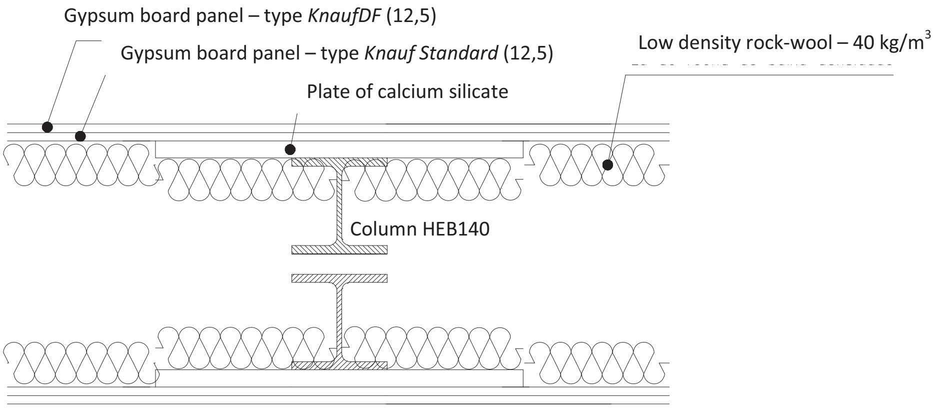


Fig. 24.2 Construction details of the wall between modules, plan view

The evaluation of fire safety requires the standardISO 834 fire from inside the building; therefore, in calculating the walls resistance, the more critical vertical element from the facade wallis the interiorvertical element. The solution for thefacade walls should be similar to the solution adopted for the inside walls: the interior vertical elementshould be equal to one of the vertical elements from the inside walls;for the exterior vertical elementthe indication of the thermal study shouldbe followed: i) 75 mm of low density rock-wool of 40 kg/m^3 (excluding the zone of the beams, in which it should be 50 mm); ii)*Aquapanel*board with 12.5 mm thick; iii) system of ventilated facades.

24.4.2 Drop ceilings

For the drop ceiling, it is recommended the application of (from bottom to top): i) 2plasterboards type*Knauf DF* with 12.5 mm thickness; ii) 50 mm of low density rock-wool of 40kg/m^3 (see Fig. 24. 3).

24.5 SELECTED DESIGN FIRE

The evaluation of fire safety requires the ISO 834 fire from inside the building.

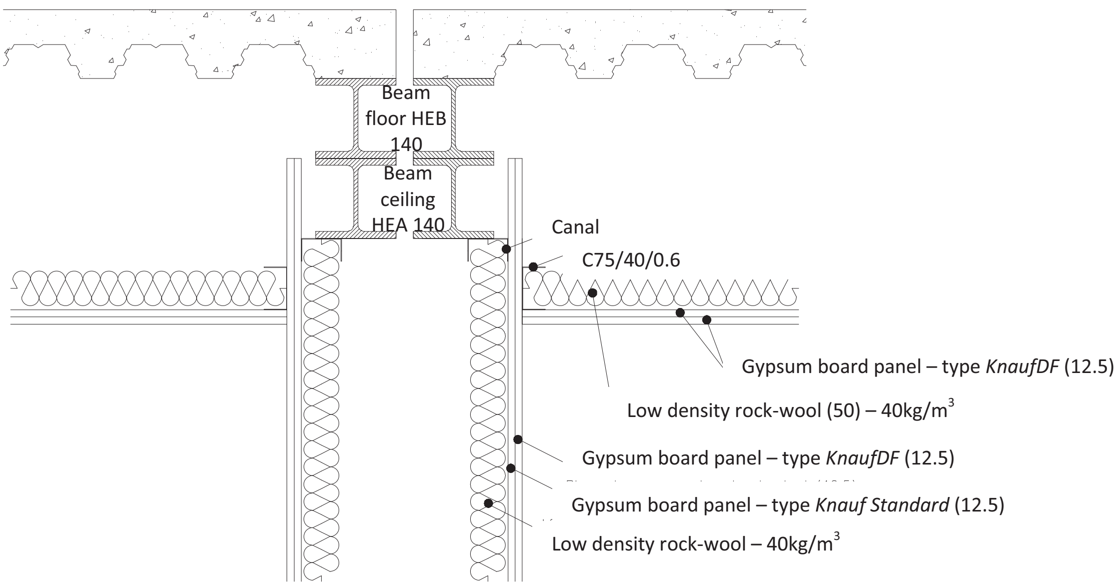


Fig. 24. 3 Construction details of the wall between modules, elevation view.

24.6 DESIGN OF STRUCTURAL ELEMENTS

24.6.1 Critical temperatures of the structural elements

The columns designis performed according to Eurocode 3, Part 1-2 (EN1993-1-2,2005),taking into account the ISO 834fire curve. The fire safety verification of the structureis made in the temperature field, imposing that the temperature does not exceed the critical temperatureduring the fire duration prescribed by the regulations (90 min.). In calculations, it is consideredthat the fire protection materials,used in the structural elements zone,sustain their insulationand resistance capabilities during 90 min. The temperature increase of an insulated steel memberduring the time interval Δt= 90 isgiven by:

$$\Delta\theta_{a.t} = \frac{\lambda_p / d_p}{c_a \rho_a} \frac{A_p}{V} \left(\frac{1}{1+\phi/3} \right) \left(\theta_{g.t} - \theta_{a.t} \right) \Delta t - \left(e^{\phi/10} - 1 \right) \Delta\theta_{g.t} \tag{1}$$

wherethe amount of heat stored in the protection is given by:

$$\phi = \frac{c_p \rho_p}{c_a \rho_a} d_p \frac{A_p}{V} \tag{2}$$

where c_p [J/kgK]is the specific heat of the fire protection material and ρ_p[kg/m^3]is the protection materialunit mass, d_p[m]is the thickness of the fire protection material; c_a[J/kgK]and ρ_a[kg/m^3]are thespecific heat and the unit mass of steel, and A_p/V is the section factor for steel members insulated by fire protection material.

The critical temperature is calculated using the expression:

$$\theta_{cr} = 39.19 \ln\left(\frac{1}{0.9674\mu_0^{3.833}} - 1\right) + 482 \tag{3}$$

where μ_0 is the degree of utilization of the element at time t = 0, and μ_0 can be obtained from:

$$\mu_0 = \frac{E_{fi,d}}{R_{fi,d,0}} \tag{4}$$

where $R_{fi,d,0}$ is the value of $R_{fi,d,t}$ for time t = 0, for 20ºC, but calculated with the safety factors related tothe fire situation and $E_{fi,d}$ is the design effectof actions for the fire design situation, given by:

$$E_{fi,d} = \eta_{fi} E_d \tag{5}$$

where E_d is the design effects of actions determined at ambient temperature, and η_{fi} is the reduction factor for design load level in the fire situation. In this calculation, the basic live loadis considered to be the most representative value from the actions:

$$\eta_{fi} = \frac{\gamma_{GA} G_k + \psi_{1,1} Q_{k,1}}{\gamma_G G_k + \gamma_{Q,1} Q_{k,1}} = \frac{1.0x(2.23 + 1.25 + 0.4x2) + 0.5 \times 2.0}{1.35x(2.23 + 1.25 + 0.4x2) + 1.5 \times 2.0} = 0.6 \tag{6}$$

The $E_{fi,d}$ values for each studied element are presented in the following tables (Tab. 24.5, and Tab. 24. 6).The ULS valuesare selected from the stability study at ambient temperature.

Tab. 24.5 Loads in the main columns at ambient and elevated temperature

Column HEB 140		$N_{,Ed}$ (kN)		$V_{z,Ed}$ (kN)		$M_{y,Ed}$ (kNm)		$V_{y,Ed}$ (kNm)		$M_{z,Ed}$ (kNm)	
		Ext. 1	Ext. 2	Ext. 1	Ext. 2	Ext. 1	Ext. 2	Ext. 1	Ext. 2	Ext. 1	Ext. 2
	ULS	363	363	6.7	6.7	20.1	2.1	1.9	1.9	-4.3	2.3
	fire	218.3	218.3	4.0	4.0	12.1	1.3	1.1	1.1	-2.6	1.4

Tab. 24.6 Loads in the main beams at ambient and elevated temperature

		$V_{z,Ed}$ (kN)		$M_{y,Ed}$ (kNm)		
		Ext. 1	Ext. 2	Ext. 1	Centre	Ext. 2
Beam - floor HEB140	ULS	39.4	-38.6	0	78.1	0
	fire	23.7	-23.2	0	47.0	0
Beam - ceiling HEA140	ULS	16.5	-16.5	0	21.3	0
	fire	9.9	-9.9	0	12.8	0

To define the critical temperatures, the following verifications are performed:

i) Columns – Buckling verification, and verification of the cross-section resistance under biaxial bending.

ii) Floor beams– Verification of the cross-section resistance to bending and axial forces. It is considered that the slab ensures the lateral restraint of the floor beams.

iii) Ceiling beams– Verification of the cross-section resistance to bending and axial forces. The lateral buckling is neglectedbecausethe loads are really small.

Tab. 24.7 presents the critical temperatures obtained for each studied element.In these calculations it is considered that all steel profiles are S275 class. Theconsidered effective length ofcolumns is l_{fi} = 0.5 x 3.32 m (the maximum compression stressesoccur in the lower floors).

Tab. 24.7 Critical temperatures

	Profile	Critical Temp.	Conditioning load
Column	HEB 140	621.9 ºC	Buckling by biaxial bending
Beam - floor	HEB 140	594.3 ºC	$M_{fi,Ed}$
Beam - Ceiling	HEA 140	701.6 ºC	$M_{fi,Ed}$

24.6.2 Columns

The columns are defined by hot rolled HEB 140 profiles.Two situations are studied: case A, the column is inserted into walls, and it is considered that the fire protection comes from the materials constituting the walls (Fig. 24.1); case B, the column is not inserted into walls and is isolated with hollow encasement.

In case A, in order to calculate the temperature evolution in columns, fire acting only on one side of the cross section (fire inside the modules) is considered (Fig. 24. 4) and it is assumed that fire protection materials maintain their insulation and resistance capabilities for 90 min. It is also considered that the fire protection is ensured by the following materials: i) calcium silicate board, type *Promatec-H,* 20 mm thick and 540 mm wide; ii) 2 gypsum board panels (the external should be of type *KnaufDF*and the internal of type *Knauf standard*), with 12.5 mm thick each.

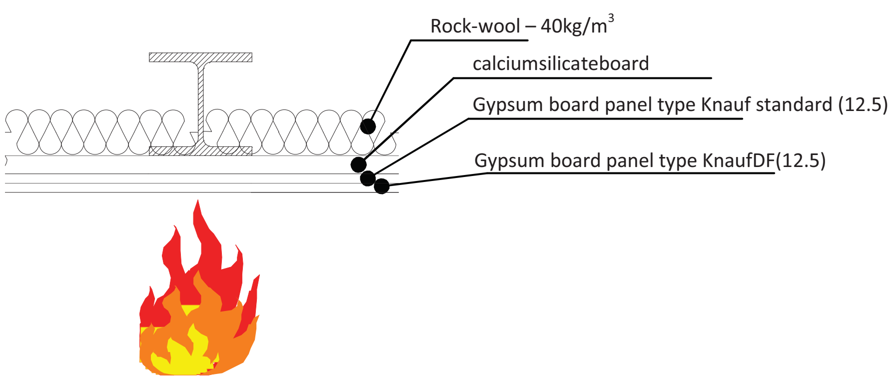


Fig. 24. 4 Representation of the fire action to the columns –Case A

The studied case B corresponds to twoisolatedprofilestogether, with hollow encasement, as showed in Fig. 24.5a). As this solution of two isolated profiles together is not directly considered in Eurocode3, Part1-2, the evolution oftemperature wascalculated for an approximatesituation: profileisolatedexposed tofireon all sideswith hollow encasement (Fig. 24.5b)). The fire protection is ensured bya calcium silicate board panel of type Promatec-H with 20 mm thick.

a) Twoisolatedprofilestogether b) Simplified situation

Fig. 24.5 Representation of the fire action to the columns –Case B

The section factor of the HEB 140 profile only exposed on one side is A_p/V = 32.59m^{-1}, and exposed on four sides with hollow encasement isA_p/V = 130.35m^{-1}. Fig. 24.6 shows the temperature evolution in the column cross section for the two cases, andthe steel temperature is lower than the critical temperature.The critical temperature of the HEB 140 profile is: 621.9°C and the steel temperature is: 111.5°C in case A, and 585.5°C in case B.

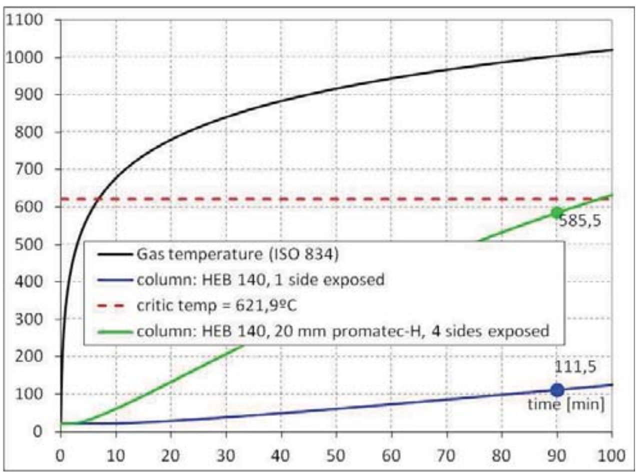


Fig. 24.6 Temperature evolution in steel column section during the ISO 834 fire

24.6.3 Interior and external beams

As for the column, the study also includesthe two study cases:case A, the beam is inserted into the ceiling; case B, the beam is out of the ceiling. The cross section of interior beams and external beams is the same, and is defined by hot rolled HEA 140 profile (ceiling beams) and HEB 140 profile (floor beams), as shown in Fig. 24. 3 (interior beams cross section).

For the case A, in order to design the beam section according to Eurocode 3, Part 1-2, the horizontal protection existing in the drop ceiling beneath the steel profiles is considered: i) 50 mm of low density rock-wool of 40kg/m^3; ii) 2 plasterboards *Knauf DF* with 12.5 mm thick (Fig. 24.7). Taking into account the position of the profiles, it is expected that the temperature of the floor beams is less than in the ceiling beams, which would lead to verify only this one; however, as the floor beams have a lower critical temperature, they are also verified in the same way: the horizontal protection existing in the drop ceiling beneath the floor beams is considered.The section factors of HEB 140 and HEA 140 profiles exposed on one side are, respectively, A_p/V= 32.59m^{-1},A_p/V = 44.59m^{-1}.

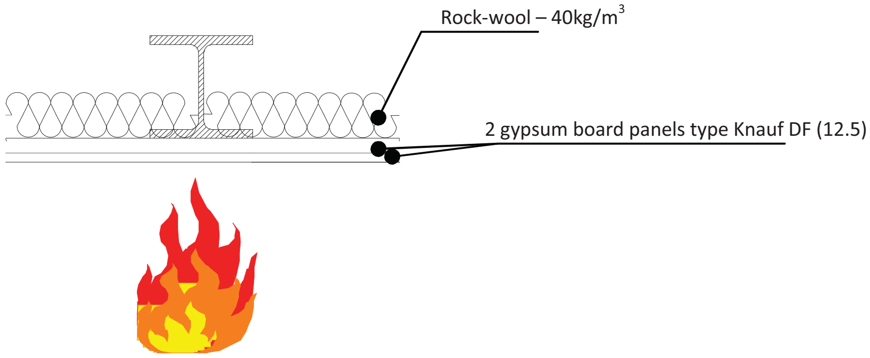
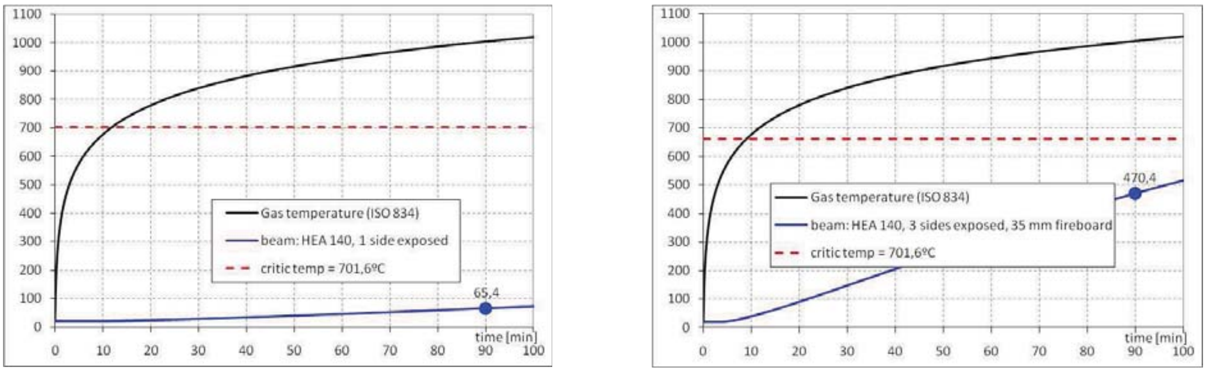


Fig. 24.7 Representation of the fire action to the beams –Case A

For the case B, it is considered that the beam resistance REI90 is ensured by the hollow encasement fire protection defined by a gypsum board panel type Fireboard, with 35 mm thick (Fig. 24.8). A few simplifications were also made to use Eurocode 3, Part 1-2: it was considered that the ceiling beams were exposed to fire on 3 sides, whereas the floor beams were not exposed to fire. The section factors of HEA 140 profiles exposed on three sides is A_p/V= 129.22m^{-1}.

Fig. 24.8 Representation of the fire action to the beams HEA 140 – Case B

Fig. 24.9 shows the temperature evolution in beams HEA profiles for the two cases A and B. For both case, at the end of 90 min., the steel temperature is lower than the critical temperature.



a) Case A b) Case B

Fig. 24.9: Temperature evolution in steel beam section HEA 140 during the ISO 834 fire

24.6.4 Slabs

The slab is "protected" by thedrop ceiling (Fig. 24. 3 – Slab cross section). The resistance of 60 min. (REI60) is guaranteed based on the materials insulation and resistance (EI60).

24.7 CONCLUSION

This paper presented the evaluation of a constructional solution developed for a modular steel structure, taking into account the fire safety design. Prefabricated modules are intended to be used for construction of hotel buildings. The results of this study are summarized in Tab. 24.8. The solution presented is in accordance withtherequired regulations, andhereafter are remembered the scope and limitations of this study:

i) The proposals for the partitions elements are based on the knowledge and experience of the University of Coimbra, Portugal, and on data of fire protection materialsfrom manufactures.

ii) The design of structural elements (beams, columns and slabs) is performed according to theEurocodes 1 and 3, Part 1-2. The calculation procedure is based on a prescriptiveapproach and is carried out in the field of temperature.

iii) In many cases, the geometrical definition of the structural profiles insulationdid not fit to the typologies indicated in Eurocodes. In these situations, simplifications and approximations,taking into account the safety of the structure, aremade to fit the problem in the Eurocodes typologies and formulations.

iv) The critical temperature is obtained by considering the steel grade S275, and the design effect of action for the fire situation is obtained from the design effect of action at ambient temperature and η_{fi}= 0,6 is used.

v) Use of advanced calculation modelscould lead to more economical solutions.

Tab. 24.8 Results summary

<table>
<tr><th></th><th>Regulatory requirements</th><th colspan="3">Study cases</th><th>θ_{cr} (ºC)</th><th>$\theta_{a,t}$ (ºC)</th></tr>
<tr><td>Walls</td><td>EI60</td><td colspan="5" rowspan="2">The proposals are based on the knowledge and experience of UC and the data of manufacturers of fire protection materials</td></tr>
<tr><td>Drop ceilings</td><td>EI60</td></tr>
<tr><td rowspan="2">Columns HEB 140</td><td rowspan="2">REI90</td><td colspan="2">Case A – columns inserted into walls</td><td></td><td>621.9</td><td>111.5</td></tr>
<tr><td colspan="2">Case B – isolated column</td><td>Promatec-H</td><td>621.9</td><td>585.1</td></tr>
<tr><td rowspan="3">Beams HEA 140 and HEB 140</td><td rowspan="3">REI90</td><td rowspan="2">Case A –beams inserted in ceiling</td><td>HEB 140</td><td rowspan="2">Knauf DF</td><td>594.3</td><td>54.3</td></tr>
<tr><td>HEA 140</td><td>701.6</td><td>65.4</td></tr>
<tr><td colspan="2">Case B – Isolated beams (HEA 140)</td><td>Knauf Fireboard (exposition 3 sides)</td><td>701.6</td><td>470.4</td></tr>
<tr><td>Slabs</td><td>REI60</td><td colspan="5">The 60 min. resistance must be ensured on the basis of the insulation and resistance of the drop ceiling insulation materials</td></tr>
</table>

<u>References</u>

DecreeLaw (DL) nº 220/2008; “Regime Jurídico de Segurança contra incêndios em edifícios”, p7903-7922.

EN 1993-1-2:2005. “Eurocode 3: Design of steel structures – Part 1-2: General rules – Structural fire design”, CEN, 78p.

NP EN 1991-1-2:2010, “Eurocódigo 1: Acções em estruturas – Parte 1-2: Acções gerais – Acções em estruturas expostas ao fogo”, 64p.

Portaria nº1532/2008; “Regulamento Técnico de Segurança Contra Incêndios em Edifícios” ,p9050-9127.

WG2- Raul ZAHARIA, raul.zaharia@ct.upt.ro

WG3- Dan DUBINA, dan.dubina@ct.upt.ro

25 CASE STUDY: FIRE DESIGN VALIDATION

Summary

The case-study presents the particular validation of the advanced calculation model SAFIR (Franssen, 2005), asked by the Romanian authorities, for the verification of the fire resistance of the composite columns of a high-rise building.

25.1 INTRODUCTION

Using the computer program SAFIR, the authors made the verification of the fire resistance for the columns of “Bucharest Tower Center” high-rise office building, situated in Bucharest, Romania (Zaharia et al., 2007). The columns are made of partially concrete encased sections, with crossed I-sections, of octagonal and rectangular shape, as shown in Fig. 25.1.

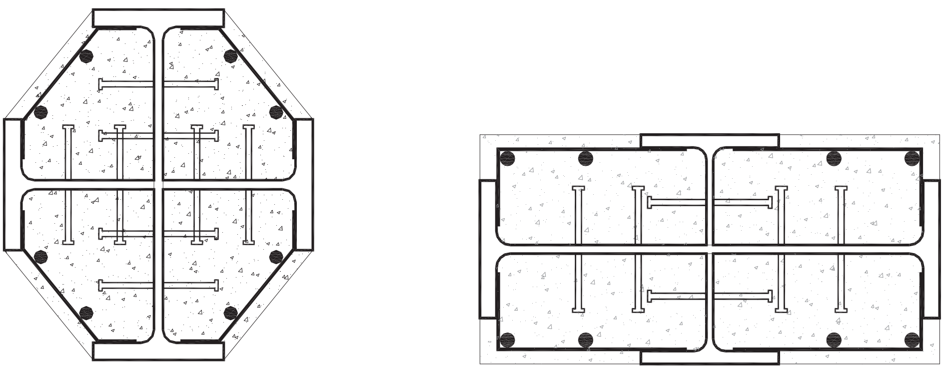


Fig. 25.1 Composite columns cross-sections

The fire resistance demand for these columns is of 150 minutes. The columns, considered as isolated elements and loaded with axial force and bending moments on both principal cross-axes (internal forces corresponding to the combination of actions for fire situation), were modelled with 3D beam elements. The buckling length of the columns was considered as the system length and equivalent imperfections according to the Eurocode specifications (EN 1994-1-1, 2005) were considered in both directions of the principal cross-section axes. The numerical analysis showed that the composite columns present a good behaviour under ISO fire; fire protection is needed only for a limited number of columns within the building, in order to attain the fire resistance of 150 minutes.

For this verification of fire resistance using an advanced calculation model, the Romanian authorities asked for a validation of the computer program SAFIR, using available experimental data of a fire test for a similar structural element. Partially concrete encased sections with crossed steel I-sections, used for the columns of "Bucharest Tower Center" building were promoted since 1987 by ARBED Luxembourg, in order to obtain high fire resistance times without supplementary fire protection (REFAO, 1987).

Therefore, the validation of the advanced calculation model SAFIR requested by the authorities considered a composite column with cross section similar to the cross sections of the "Bucharest Tower Center" columns, from the experimental report REFAO (1987), which presents some fire tests conducted on composite beams and columns made by steel profiles embedded in concrete.

25.2 FIRE TEST AND VALIDATION

The experimental specimen from the fire test considered for the validation, subjected to compression, is shown in Fig. 25.2 (REFAO, 1987). The test was conducted under standard ISO temperature-time curve. A fire resistance of 172 minutes was obtained, without supplementary protection for the profiles flanges.

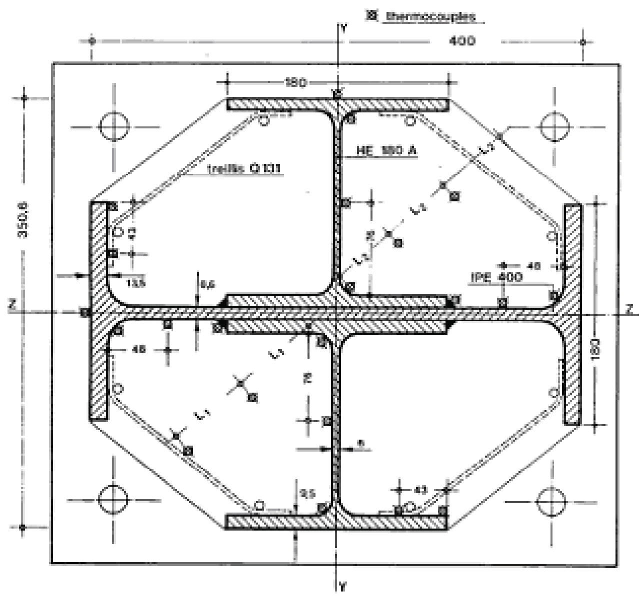


Fig. 25.2 Cross-section of the composite column tested under ISO fire (REFAO, 1987)

Fig. 25.3 shows the comparison between the measured temperatures within the composite cross section of the column (REFAO, 1987) and the temperatures obtained from the numerical analysis, at the failure time of the tested column of 172 minutes. It has to be mentioned that the resultant emissivities considered in the numerical model were the ones given in the test report for the surfaces of steel elements (0.3 and 0.5) and for the concrete (0.45). These values are lower than the surface emissivities given in EN1994-1-2 for steel and concrete (0.7), which were used for the fire resistance assessment of "Bucharest Tower Center" columns.

The numerical thermal analysis offers good results, with values which are close to the calculated temperatures, in the points from the cross section where the thermocouples were placed.

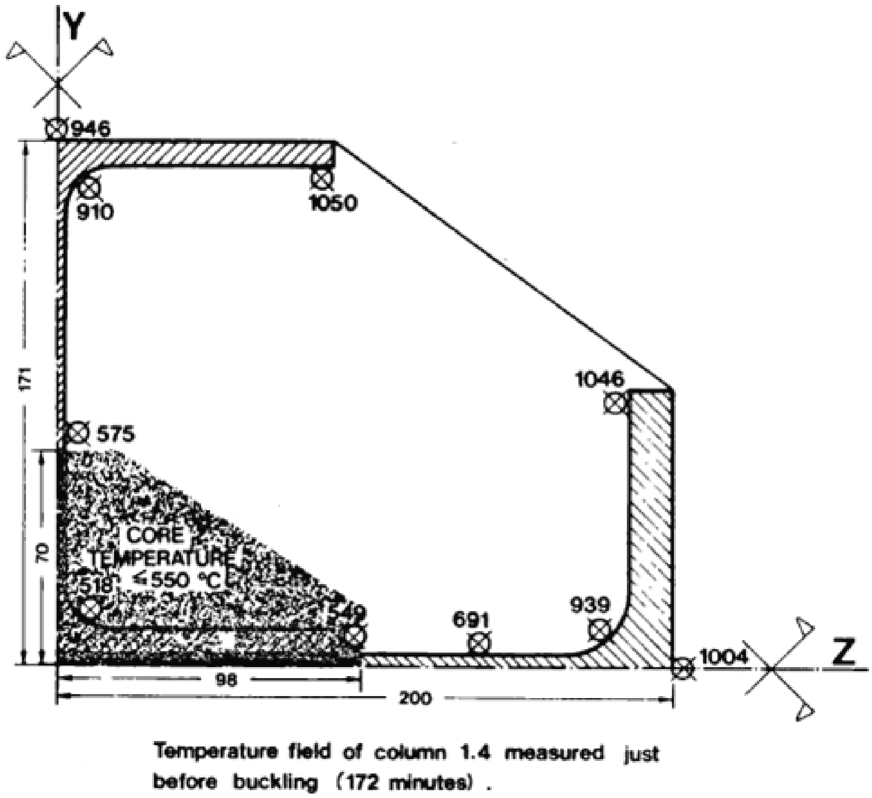
Considering a buckling length equal to the length of the column (both ends are pinned, as supposed to be in the test) the failure time given by the numerical analysis is of 132 minutes. This is a very conservative result, in comparison with the failure time of the experimental specimen of 172 minutes.

If a buckling length of 50% from column length is considered (both ends are fixed), the failure time obtained by numerical analysis is of 188 minutes, higher than the failure time obtained in the test. If a buckling length of 70% from column length is considered (intermediate situation between fixed and pinned supports), the failure time given by the numerical analysis is of 164 minutes. This suggests that for the tested column it was not possible to realize perfect pinned ends and there was a degree of rotational restraint at the supports.

The values of fire resistance times presented above were determined for a small initial global imperfection of 1 mm introduced in the numerical analysis, even if the test report states that no initial imperfection was emphasised after measurements.

25.3 CONCLUSIONS

SAFIR offers good results, in the safe side, in comparison with the results obtained from the fire test. Consequently, the advanced calculation model SAFIR was validated for the particular type of composite columns of "Bucharest Tower Center" building.



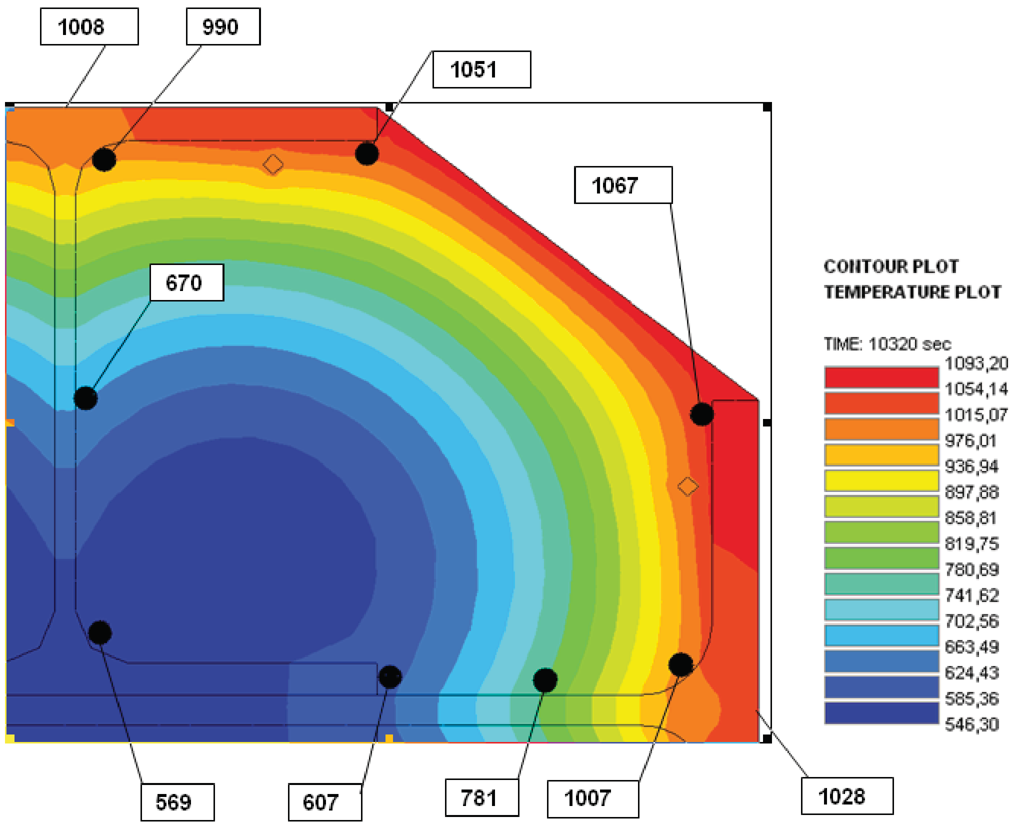


Fig. 25.3 Temperature distribution on cross-section:

experimental (REFAO, 1987) and numerical

References

EN 1994-1-1 (2005), "Eurocode 4 – Design of composite steel and concrete structures. Part 1-1. General rules and rules for buildings", CEN, Brussels.

EN 1994-1-2 (2005), "Eurocode 4 – Design of composite steel and concrete structures. Part 1-2. General rules – Structural Fire Design", CEN, Brussels.

Franssen J. M. (2005), Safir – A thermal/structural program modelling structures under fire, Engineering Journal, AISC, Vol. 42, No 3, 143-158.

REFAO (1987), C.E.C. Agreement Number N° 7210-SA/502, Computer assisted analysis cf the fire resistance of steel and composite concrete-steel structures (REFAO - CAFIR), Commission of the European Communities, Directorate-General, Science, Research and Development, EUR 10828 EN, ECSC-EEC-EAEC, Brussels – Luxembourg.

Zaharia, R., Pintea, D., Dubina, D. (2007), "Fire analysis and design of a composite steel-concrete structure", Proceedings of the 3rd International Conference on Steel and Composite Structures - ICSCS07, Manchester, UK, 30 July-1 August, Taylor & Francis/ Balkema, Editors: Y. C. Wang & C. K. Choi, ISBN978-0-415-45141-3, 725-730.

M. Jones, mark.jones@exova.com

J. Ding, summer.ding@exova.com

26 STRUCTURAL FIRE ANALYSIS: BLESSED TRINITY RC SCHOOL, BURNLEY

Summary

Fire engineering analysis has been carried out on a proposed three storey secondary school in order to demonstrate that the building will fulfill the functional requirement of the UK Building Regulations with regard to structural fire protection, with a reduced overall standard and coverage of said protection than that specified in the code. The principle outcome for the project contractor was decreased costs in terms of the amount of intumescent paint required and the associated costs of transportation and application. The analyses were carried out with the full consent of relevant stakeholders and approvals authorities following a detailed qualitative design review. The building is now complete, occupied and functioning as intended. The analysis consisted of determining the baseline equivalent fire resistance based upon likely fire severity in accordance with BS 7974 principles. A full frame finite element analysis was then undertaken to demonstrate that further fire protection could be removed from secondary structure without comprising the required structural stability.

26.1 SUMMARY

This chapter summarises in 17 pages the results of a fire engineering study which has been carried out to determine the level of structural fire resistance required to a sprinklered steel-composite framed building. It has been shown that a reduced 30 minute standard to main structural elements will fulfil the functional requirement of the UK Building Regulations without the need to protect much of the secondary structure given the characteristics of the design.

26.2 DESCRIPTION OF BUILDING

The building is a three-storey new build school in Burnley, Lancashire which will provide educational facilities for up to 1250 students. The building is to be constructed on a sloping site with the main entrance to the school located on Level 2 (upper ground floor). In addition to the general teaching areas at all levels, Level 1 provides dining and kitchen accommodation and a library, Level 2 has a theatre and stage area as well as a café and servery.

The building will have a sprinkler suppression system designed and installed in line with the current British Standard. The installation of the sprinkler system allows flexibility of design in terms of compartmentation and open spatial planning, facilitating the aspirations of the client and the Architect in terms functionality. The structure of the building is a three storey composite steel frame. Composite slabs are used in the building, 150mm thick reinforced concrete slab on multideck 50. All the beam-column connections and beam-beam connections are designed to be pinned joints.

26.3 REGULATORY REQUIRMENTS

The relevant regulatory requirements for the building are the UK Building Regulations. With regard to the fire protection of loadbearing elements of structure, the relevant Requirement B3 of the Building Regulations states, "The building shall be designed and constructed so that, in the event of a fire, its stability will be maintained for a reasonable period." To comply with Approved Document B (ADB), all loadbearing elements of structure should be provided with a minimum of 60 minutes fire resistance, given the height of the building and that the building will be fully sprinklered. Any structure which supports only the roof does not need to be fire resisting unless the structure is essential for the stability of an external wall which needs to have fire resistance. However the ADB recognises that alternative 'fire engineered' approaches may be used to achieve compliance with the Building Regulations. This provides an opportunity for a performance-based design approach.

26.4 FIRE ENGINEERED APPROACH

The fire engineered assessment of this building adopts a two-pronged strategy. First, the anticipated fire conditions within given enclosure are characterised with reference to a set time duration of the standardised gas temperature/time relationship described in BS 476 Part 20. The duration of exposure is derived empirically and referred to as the equivalent time of fire exposure or 'time equivalent' value. The time equivalency calculation method, described in PD 7974 Part 3, can be used to directly relate the severity of a real fire to an equivalent period of a standard fire resistance test, based upon the enclosure details, i.e. fire load, amount of ventilation, and thermal properties of the boundary materials. Safety factors are also incorporated into the calculation to take into account the likely risks and consequences of a fire. The intention is to carry out an analysis of the 'reasonable worst case' fire in each individual area to determine its impact on the structure. Thus the baseline level of fire protection required for loadbearing elements of structure may be determined.

Following the time equivalency analysis a full frame finite element analysis is carried out with the baseline level of fire protection eliminated from secondary structural members. The intention is to show that the steelwork within each compartment (columns and beams with slabs above) will not fail when exposed to BS476 standard fire conditions for the minimum required fire resistance period. Vulcan software is used to carry out the finite element modelling.

26.5 PERFORMANCE/ASSESSMENT REQUIREMENTS

As the building is sprinklered throughout, it is assumed to be effectively compartmented on a floor by floor basis (i.e. a fire is unlikely to spread to more than one floor). Due to the high fire load in the library, it is proposed to enclose the library and its associated area in a separate compartment that achieves 60 minutes fire resistance. In addition to the library compartment, each floor of the building is divided into 5no compartments. Areas of special fire hazard will be enclosed in construction achieving 30 minutes fire resistance. Protected stairs will be enclosed in construction achieving at least 30 minutes fire resistance. It is assumed that a fire may involve a complete compartment, but will not spread to other compartments. Therefore it is reasonable to assess one fire compartment at a time in each phase of the analysis. Compartment layouts are shown in Fig. 26.1 – 26.3.

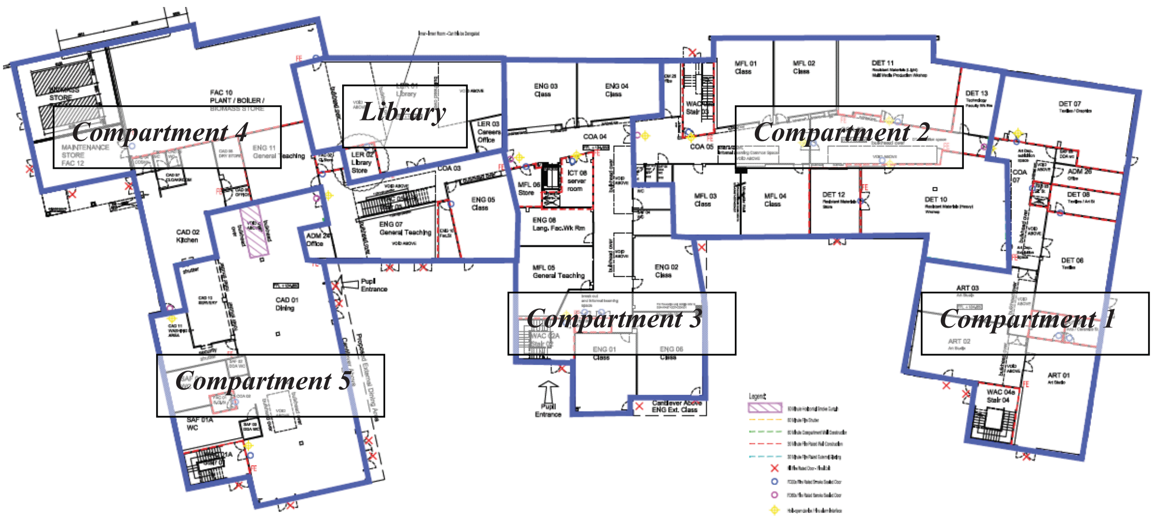


Fig. 26.1 Fire compartments on Level 1

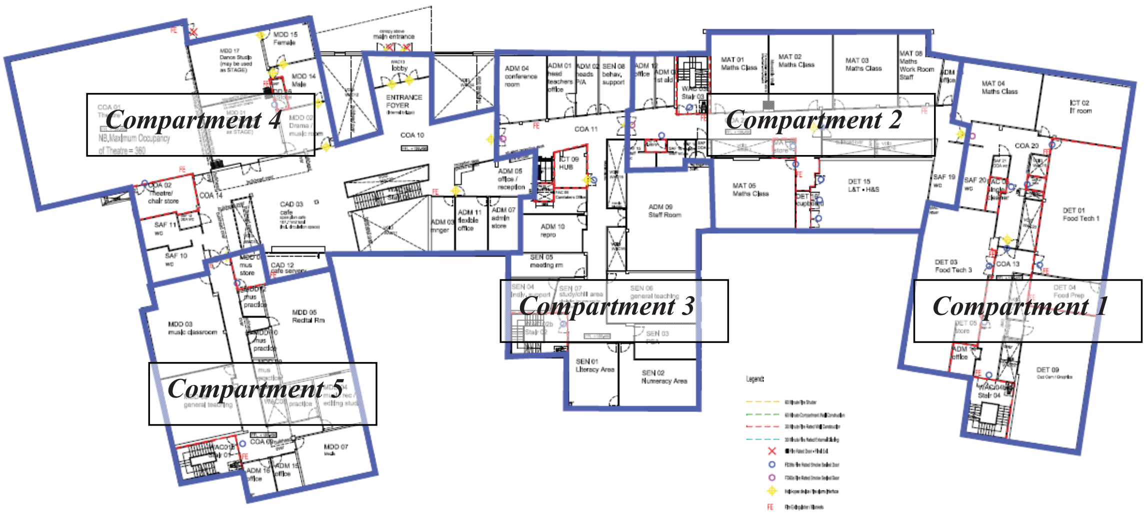


Fig. 26.2 Fire compartments on Level 2

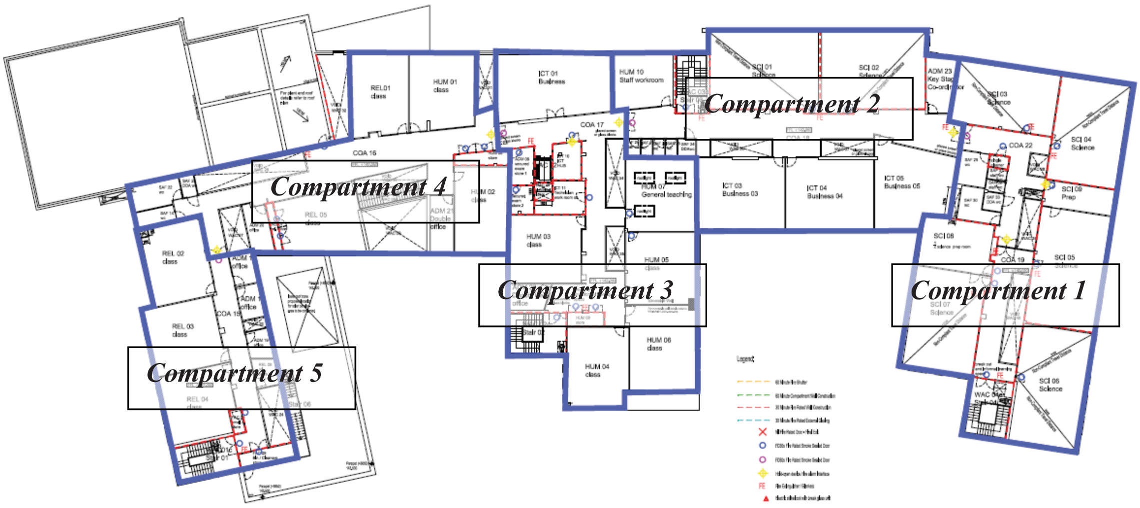


Fig. 26.3 Fire compartments on Level 3

26.5.1 Phase 1 Time Equivalence

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

26.5.2 Phase 2 Vulcan Analysis

Each sub-frame representing a compartment will be exposed to BS 476 standard fire for the required fire resistance period identified from Phase 1 or 30 minutes for places of special fire hazard. According to ADB, beams and columns should maintain the loadbearing capacity for the required fire resistance period when tested to BS 476. BS 476 Part 20 states that failure of a column to maintain its loadbearing capacity shall be deemed to have occurred when it fails to support the load. This can be checked by the finite element analysis as the finite element model stops once any one structural element loses its loadbearing capacity. The maximum deflection of a beam should be limited to $L/20$ where L refers to the clear span. This limit is also applied to steel beams without external fire protection. ADB requires floors to maintain loadbearing capacity, integrity and insulation for the required fire resistance period. As for the beams, a slab shall be deemed to have failed if it is no longer able to support the load with a maximum allowed deflection of $L/20$ as required by BS 476. According to BS 476 Part 20, failure to maintain integrity shall be deemed to have occurred when collapse or sustained flaming on the unexposed face occurs or the criteria for impermeability are exceeded. Impermeability is normally satisfied for composite slabs. Should a slab collapse the finite element model will then stop. For composite slabs, insulation normally is not a problem.

Due to the nature of composite steel-framed buildings, localized failure (a limited number of structural elements losing stability) is unlikely to cause the whole frame to collapse. However, should the model stop before the required fire resistance period, it is considered that the structural frame does not meet the fire resistance requirements.

26.6 TIME EQUIVALENCE ANALYSIS

26.6.1 Fire Load

In order to determine the heat release rate of a fully involved compartment fire, it is necessary for the fire load within the compartments to be determined. PD 7974-1 provides statistical design data for typical fire load densities within various occupancies. However, it is noted in PD7974-1 that "the fire load densities assume perfect combustion, but in real fires, the heat of combustion is usually considerably less". Therefore the 80% fractile value is taken as the fire load density for design. This is the average value that is not exceeded in 80% of rooms or occupancies.

In accordance with Table A.19 in PD7974-1, the 80% fractile fire load density for school buildings is 360MJ/m^2; the 80% fractile fire load density for libraries is 2250 MJ/m^2. As the fire load density of libraries is much higher than normal school areas, it is recommended to enclose the library in a separate fire compartment to 60 minutes standard and apply 60 minute protection to elements of structure in this location.

26.6.2 Ventilation to the enclosure

Ventilation is accounted for in the time equivalency method by calculating a 'ventilation factor', w_v. This factor is based upon the size and orientation of the vents/openings, the floor area and the height of the enclosure and is taken from PD7974-3.

$$w_v = 1.7H^{-0.3}\left\{0.62 + 90\left(0.4 - {A_v}/{A_f}\right)^4\right\}\left(1 + b_v\, A_h/A_f\right)^{-1} \geq 0.5 \tag{1}$$

$$b_v = 12.5\left\{1 + 10\left({A_v}/{A_f}\right) - \left({A_v}/{A_f}\right)^2\right\} \geq 10 \tag{2}$$

w_v= ventilation factor

H = Height of the enclosure (measured from floor to ceiling)

A_f = Floor Area

A_h = Area of ventilation in the horizontal plane

A_v = Area of ventilation in the vertical plane

b_v = Calculation factor

When assessing the amount of ventilation to the enclosure, the following additional assumptions have been made:

- Testing has shown that the critical temperature for standard toughened glass is around 200°C. Therefore, it is generally assumed that all the glass without fire resistance will break shortly after fire ignition. However, a sensitivity study has been carried out to investigate the effect of glazing breakage. This analysis investigated both 75% and 100% of the glazing area of the enclosure breaking.
- The analysis is based upon perfect combustion for the duration of the fire, i.e. there is no growth period.

26.6.3 Thermal properties of the bounding materials

The heat loss to the structure and thermal properties of the bounding elements, e.g. walls, floors, ceiling, etc, are taken into account by the factor k_b. Values of k_b are given in Tab. 26.4 of PD 7974-3 and are based upon the thermal inertia of the materials. The bounding materials used in this building are typical building surfaces, such as glazing, gypsum plaster and concrete which have a typical thermal inertia of between 720-2500 J/m^{2}s$^{1/2}$K. This gives a value of 0.07 for k_b.

26.6.4 Equivalent time of fire exposure, t_e

The equivalent time of standard fire exposure can be defined as:

$$t_e = k_b w_v q \tag{3}$$

t_e = time equivalent value (min)

k_b = factor relating to the thermal properties of the enclosure

w_v = ventilation factor

q = fire load density (MJ/m^2)

26.6.5 Design Time Equivalent Duration, t_{ed}

While Equation 3 is a reasonable predictor of enclosure fire conditions, it does not consider those less quantifiable influences on the design fire duration. Therefore, various safety factors are added to take into account the building height, occupancy type and provisions of automatic sprinkler systems. The design time equivalent duration, t_{ed} can be defined as:

$$t_{ed} = \gamma_1 \gamma_2 \gamma_3 t_e \tag{4}$$

γ_1 = safety factor reflecting the consequences of failure of the enclosure

γ_2 = safety factor reflecting the risk of a fully developed fire taking place

γ_3 = safety factor reflecting the benefits of installing active fire fighting measures

From PD7974-3, γ_1 is based upon the height of the enclosure where for an assembly building less than 20m high, γ_1 has a value of 0.8. PD7974-3 specifies that safety factor γ_2 for assembly buildings has a value of 0.8 and recommends safety factor γ_3 of 0.6 where an automatic sprinkler system is provided.

26.6.6 Design Time Equivalence Results

The design time equivalence results are tabulated in Table 1 (compartments designated DET represent small 30 minute rooms of special fire risk). The results show that the worst case is fire compartment 1 on Level 3 which gives a design time equivalent duration of 27.5 minutes. The classrooms in this compartment are all enclosed in 30 minutes fire rated enclosure. Therefore only the corridor and WCs are included in the floor area and there is no ventilation to this area. This is onerous as the fire load in corridor and WCs is considerably low.

The results shown in Tab. 26.1 are considered onerous given that:

- The analysis is based upon the maximum burning rate for the duration of the fire, which does not take into account the growth stage of the fire;
- The fire is assumed to be under perfect combustion. In reality, this would be unlikely and the compartment temperatures would be less;
- In the fires that involve more than one room in the case of cellular plan layout, the time taken to burn through the dividing wall has not been taken into account. Although they have no defined fire resistance, there will be an inherent delay in fire spread;
- Areas such as corridors and WCs with very low fire risk are assumed to have the same fire load density;
- Automatic fire suppression systems will control the growth and spread of a fire. Accordingly, fires can be controlled within an area of burning consistent with the spatial configuration of the suppression system.

Therefore any fire will likely be contained within a small area of fire origin due to the combination of walls, floors and sprinklers. It is very unlikely that the whole compartment would be involved in the fire. In addition, assuming the sprinkler system achieves its intended function of controlling the fire size, the temperatures within the compartment can generally be assumed to be limited to 1000°C (unprotected steel maintains its strength up to 400°C).

The analysis shows that 30 minutes fire resistance to elements of structure would be sufficient, except 60 minutes for the library compartment. The walls and floors separating different compartments should also have 30 minutes fire resistance, except 60 minutes for the ones enclosing the library.

Tab. 26.1 Design Time Equivalence Results

Fire Scenario	Required Fire Resistance Period (mins)
Level 2 Compartment 1	16.29
DET03	30
DET03 01 & 04	30
Level 2 Compartment 2	15.76
DET15	30
Level 2 Compartment 3	15.9
Level 2 Compartment 4	21.86
Level 2 Compartment 5	19.3
Level 1 Compartment 1	12.67
DET07	30
Level 1 Compartment 2	18.86
DET11	30
DET10	30
Level 1 Compartment 3	14.75
Level 1 Compartment 4	15.65
Plant Room	30
Kitchen	30
Level 1 Compartment 5	12.46

26.7 FINITE ELEMENT ANALYSIS VULCAN

26.7.1 Software

Vulcan is a finite element analysis (FE) program capable of modelling the global 3-dimensional behaviour of composite steel-framed buildings under fire conditions. The analysis includes geometrical and material non-linearity with full membrane action in the slabs and cracking in concrete. Standard stress-strain curves and full thermal expansion characteristics are incorporated as functions of temperature for both steel and concrete, with uniform or non-uniform temperature distributions. The orthotropic nature of composite deck slabs is represented using an effective-stiffness concept. Vulcan output has been extensively validated against a range of data including test results of fire test programmes at the BRE Cardington facility.

The structure is modelled as an assembly of finite beam-column, spring, shear connector and slab elements. It is assumed that the nodes of these different types of element are defined in a common fixed reference plane, which coincides with the mid-surface of the concrete slab element. The beams/columns are represented by 3-noded line elements. The cross-section of each element is divided into a number of segments to allow variations in the temperature, stress and strain through the cross-section to be represented. Both geometric and material non-linearities are included. To represent steel-to-steel connections in a frame, a 2-noded spring element of zero length, with the same nodal degrees of freedom as a beam-column element, is used. The interaction of steel beams and concrete slabs within a composite floor is represented using a linking two-noded shear-connector element of zero length, with three translational and two rotational degrees of freedom at each node. The analysis includes geometric non-linearity in the slabs, using a quadrilateral 9-noded higher-order isoparametric element using an effective stiffness model to model the ribbed nature of typical composite slabs. The temperature and temperature dependent material properties can be specified independently. A maximum-strain failure criterion has been adopted for the concrete, and a smeared model has been used in calculating element properties after cracking or crushing. After the initiation of cracking in a single direction, concrete is treated as an orthotropic material with principal axes parallel and perpendicular to the cracking direction. Upon further loading of singly cracked concrete, if the tensile strain in the direction parallel to the first set of smeared cracks is greater than the maximum tensile strain then a second set of cracks forms. After compressive crushing, concrete is assumed to lose all stiffness. The uniaxial properties of concrete and reinforcing steel at elevated temperatures, specified in Eurocode 4: Part 1-2 have been adopted in this software.

26.7.2 Membrane Action

Membrane action of concrete floor slabs is due to the development of in-plane forces within the depth of the slab. Depending on the horizontal restraint conditions around the slab's perimeter, membrane action can occur at small and large vertical displacements. During a fire large displacements of floor slabs within a building are acceptable provided structural collapse or compartmentation failure does not occur. These large displacements lead to tensile membrane action occurring within the slab, which can be beneficial to the survival of the building. For slabs that have no horizontal restraint around their perimeter, membrane action can still develop provided the slab is two-way spanning.

26.7.3 Finite Element Models

The purpose of this analysis is to demonstrate that each fire compartment (sub-frame) meets the required fire resistance period identified in Phase 1. Each compartment will be modelled separately. In order to simulate the interaction between heated structure and cool structure and to model the loads accurately, a bay of sub-frame within the compartments adjacent to the fire compartment is also included in the model. All the columns above and under the compartment floor are included.

26.7.4 Material Properties

All of the steelwork in the building is grade S355. The concrete used is C28/35 (minimum) with characteristic cube strength of 35N/mm^2. Cold worked reinforcement bars with a characteristic strength of

460N/mm^2 are used in the slabs. The tensile strength of concrete is assumed to be zero. The temperature dependent material properties are taken into account in the finite element models.

26.7.5 Modelling slabs

The composite slabs used in the building are constructed on steel decking multideck 50-V2 with a total depth of 150mm and a layer of minimum A142 mesh in top (a total of 142mm^2 of mesh per metre length of slab in each direction, with 25mm cover). Full composite action between the beams and slabs is assumed. To be conservative, only the continuous depth of the slabs has been considered being 100mm thick. Ignoring the bottom 50mm of the slabs results in higher temperature, lower strength and therefore higher deflection. The contribution of the steel decking is also ignored. Instead of using a discrete series of bars, Vulcan considers the mesh as consisting of two smeared layers, reaching across the length of the slab and spanning in opposite directions. To conform to the properties of the 'true' mesh, the thickness of each layer is calculated such that they have an area of 142mm^2/m (i.e. each layer is 0.142mm thick). The slab cross-section is divided into 10 layers. Layering is important as it allows the temperature distribution within the slab to be modelled accurately.

26.7.6 Connections

All the beam-column connections and beam-beam connections are designed to be pinned joints. Spring elements have been used to simulate these connections. The springs are assumed to have indefinite axial stiffness but are free to rotate.

26.7.7 Effects Of Fire

The effects of fire on the structure are included in the models by defining the temperature profile of structural elements. The elements in the fire compartment are assumed to be heated under BS 476 standard furnace fire condition while the elements outside the fire compartment remain ambient temperature (20°C). Two aspects of temperature need to be defined: the time-temperature relationship, and the temperature profile through individual sections. For the analysis the steel frame in each compartment is initially modelled without any fire protection, except the main beams and columns that fall within 30 minute rooms which are protected to 30 minutes standard. If the model stops due to localized failure (a limited number of structural elements lose stability) at a lower time than the time equivalency then certain columns and/or beams are protected and the model is re-run. This procedure is repeated until the model has sufficient protection to comfortably exceed the time equivalency value. The selection of which columns and/or beams are to be protected is made by inspecting the behaviour of the models.

26.7.8 Thermal Response of Structural Steelwork

The performance of structural elements in fire depends upon the way in which they are heated, on the temperatures reached, on the materials used and on the way they are stressed. The method of determining the thermal response of the structural steel members is described in PD 7974-3. Uniform temperature distribution over the cross-section is assumed for steel beams and columns. For an equivalent uniform temperature distribution in the cross-section, the increase of temperature ΔT in an unprotected steel member during a time interval Δt should be determined from:

$$\Delta T = \frac{H_p / A}{c_a \rho_a} q_{net} \Delta t \qquad (5)$$

Hp/A is the section factor for an unprotected steel member (m-1),

q_{net} is the net heat flux (W/m^2),

c_a is the specific heat of steel (J/kgK),

ρ_a is the unit mass of steel (kg/m^3),

For an equivalent uniform temperature distribution in the cross-section, the increase of temperature ΔT in a protected steel member during a time interval should be determined from:

$$\Delta T = \frac{\lambda_i}{d_i} \frac{H_p / A}{c_a \rho_a} \frac{(T_g - T_a)}{(1 + \Phi / 3)} - (\exp(\Phi / 10) - 1)\Delta T_g \qquad (6)$$

with

$$\Phi = \left(\frac{c_i \rho_i}{c_a \rho_a} \right) d_i \left(\frac{H_p}{A} \right) \qquad (7)$$

where

T_g is the flame temperature (°C),

T_a is the steel temperature (°C),

λ_i is the thermal conductivity of the fire protection material (W/mK),

d_i is the thickness of the fire protection material (m),

c_i is the specific heat of the fire protection material (J/kgK),

ρ_i is the unit mass of the fire protection material (kg/m^3).

The steel temperature at time t is determined from the steel temperature at the previous time step, plus the temperature increase ΔT during the previous time increment. Steel beams are assumed to expose to fire on three sides while columns on all four sides. Protected beams and columns have been assumed to reach 550°C at 30 minutes.

26.7.9 Thermal Response of Floor Slabs

Slabs are considered to be heated only on the bottom. Slab temperatures are calculated using the built-in function of Vulcan (Eurocode 4: Part 1.2 method). As the bottom 50mm of the slab has been ignored in the models, the calculated temperatures are relatively higher than they should be.

26.7.10 Boundary Conditions

Boundary conditions are applied to the top and bottom of the columns only. The top of the columns is permitted to move only in a vertical direction so that superstructure loading may be applied. The bases of the columns are assumed to be completely fixed in place.

26.7.11 Loading

The structural effects of a fire in a building, or part of a building, should be considered as a fire limit state. A fire limit state should be treated as an accidental limit state. In checking the strength and stability of the structure at the fire limit state, the loads should be multiplied by the relevant load factors. These are given in BS 5950-8. Loadings were supplied by the structural engineers.

26.8 RESULTS OF FULL-FRAME ANALYSIS

The finite element model stops once any one structural element loses stability. Although localized failure (a limited number of structural elements lose stability) is unlikely to cause the whole frame to collapse, it is considered that the structural frame loses stability at the time the model stops. Tab. 26.2 compares the analysis results with the required fire resistance period for each fire compartment. It can be seen that given the defined fire protection every compartment maintains the loadbearing capacity for the required fire resistance period with additional safety margin. An indicative mark-up of the fire protection required to the steelwork in one area is shown in Fig. 26.6. It may be seen that the library structure remains as 60 minutes, but much of the secondary protection elsewhere may be eliminated.

As a result of the finite element modelling, the floor deflections can be output as graphed for example in Fig. 26.4. It can be seen from the figures that the maximum floor deflection in each compartment is less than $L/20$ at the end of the analysis except for Level 1 compartment 2. In Level 1 compartment 2 the $L/20$ limit is reached at 28 minutes while the required fire resistance period for this compartment is only 18.86 minutes. Therefore, the loadbearing capacity and integrity of the whole structure is maintained when exposed to a foreseeable fire. ADB states that compartment walls should be able to accommodate the predicted deflection of the floor above by either: a) having a suitable head detail between the wall and the floor, that can deform but maintain integrity when exposed to fire; or b) the wall may be designed to resist the additional vertical load from the floor above as it sags under fire conditions and thus maintain integrity.

Tab. 26.2 Summary of finite element modelling results

Fire Scenario	Required Fire Resistance Period (mins)	FE analysis results
Level 2 Compartment 1	16.29	Model stopped at 20.16 minutes
DET03	30	Model ran for 30 minutes without failure
DET03 01 & 04	30	Model ran for 30 minutes without failure
Level 2 Compartment 2	15.76	Model stopped at 19.81 minutes
DET15	30	Model ran for 30 minutes without failure
Level 2 Compartment 3	15.9	Model stopped at 18.08 minutes
Level 2 Compartment 4	21.86	Model stopped at 25.89 minutes
Level 2 Compartment 5	19.3	Model stopped at 20.91 minutes
Level 1 Compartment 1	12.67	Model stopped at 15.53 minutes
DET07	30	Model ran for 30 minutes without failure
Level 1 Compartment 2	18.86	Model ran for 30 minutes without failure
DET11	30	Model ran for 30 minutes without failure
DET10	30	Model ran for 30 minutes without failure
Level 1 Compartment 3	14.75	Model stopped at 16.05 minutes
Level 1 Compartment 4	15.65	Model stopped at 16.81 minutes
Plant Room	30	Model ran for 30 minutes without failure
Kitchen	30	Model ran for 30 minutes without failure
Level 1 Compartment 5	12.46	Model stopped at 14.59 minutes

26.8.1 Connections

Although connections are critical members in structures, it is only now that they are receiving enough attention to achieve better understanding of their behaviour in fire. The reason of not requiring assessment of connection behaviour in fire in design is based on the argument that in the connection region, the fire exposed area is low compared with the mass, thereby slowing down temperature rises in the connection region in fire compared with the connected beams and columns. However, recent observations from real fires show that, on some occasions, the accumulative effects of a number of factors (including hogging bending moment, tension field action in shear and high cooling strain or pulling in effect at large deflections of the connected beam) could make the tension components of the connections fracture. In general, steel beams restrained by the adjacent structure would go through a number of phases in behaviour when exposed to fire, including combined flexural bending and compression during the early stage of fire exposure, as a result of restrained thermal expansion of the beam, through flexural bending at temperatures around the beam's limiting temperature under pure bending, and finally to catenary action at the late stage of fire exposure when the beam deflection is very large. The compression forces in the connections are caused by restrained thermal expansion of the beams while the tension forces are caused mainly by the contraction of the beam under large deflection. The connection spring elements are assumed to have indefinite axial stiffness but free to rotate. Fig. 26.5 shows the axial forces in typical connections during heating extracted from the finite element models. By assuming indefinite axial stiffness in the connections, the deformations of the connections are neglected. When exposed to fire, steel gradually loses stiffness and strength. As such at elevated temperature the axial stiffness of the connections will also reduce which allows the beams to expand and contract to some extent. As a result the axial forces in the connections will be lower than the calculated. Nevertheless the calculated results still indicate the trend of the connection forces. It can be seen from Fig. 26.5 that the connections are still under compression at the end of the fire exposure. Therefore connection fracture is unlikely to occur.

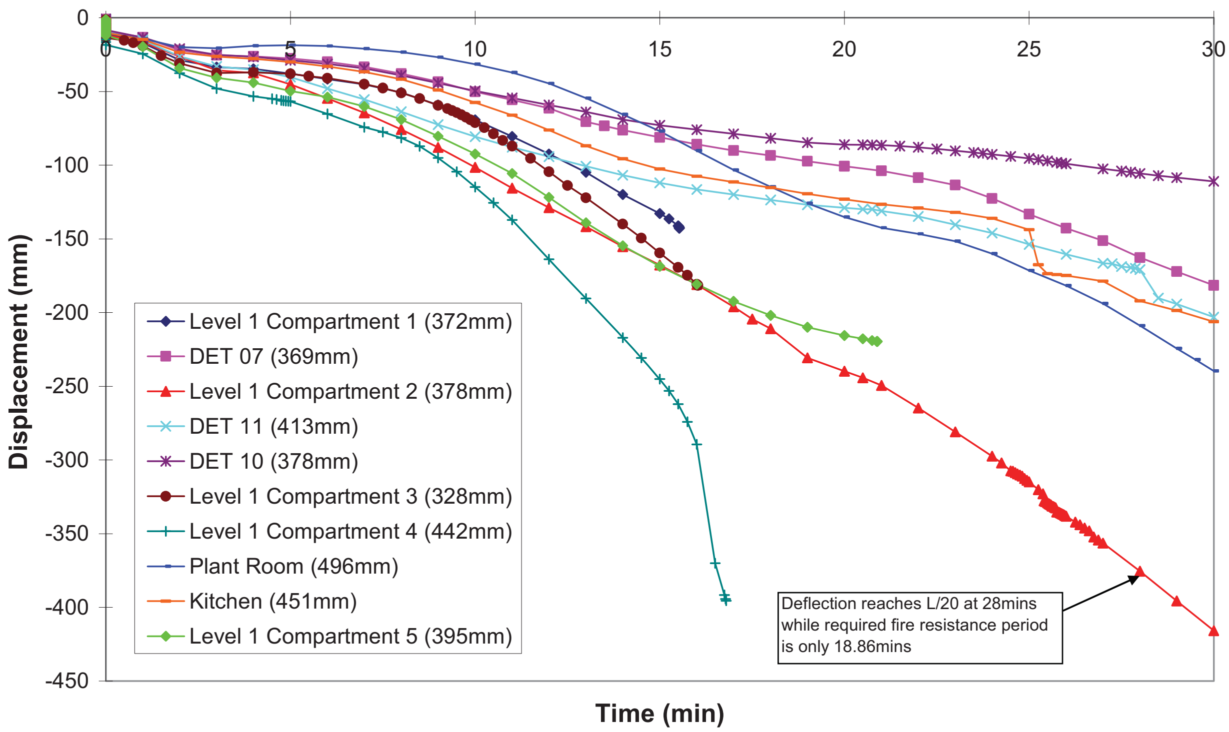


Fig. 26.4 Maximum deflection of Level 2 slabs while exposed to fires on Level 1

(max. allowable deflection L/20 indicated in brackets)

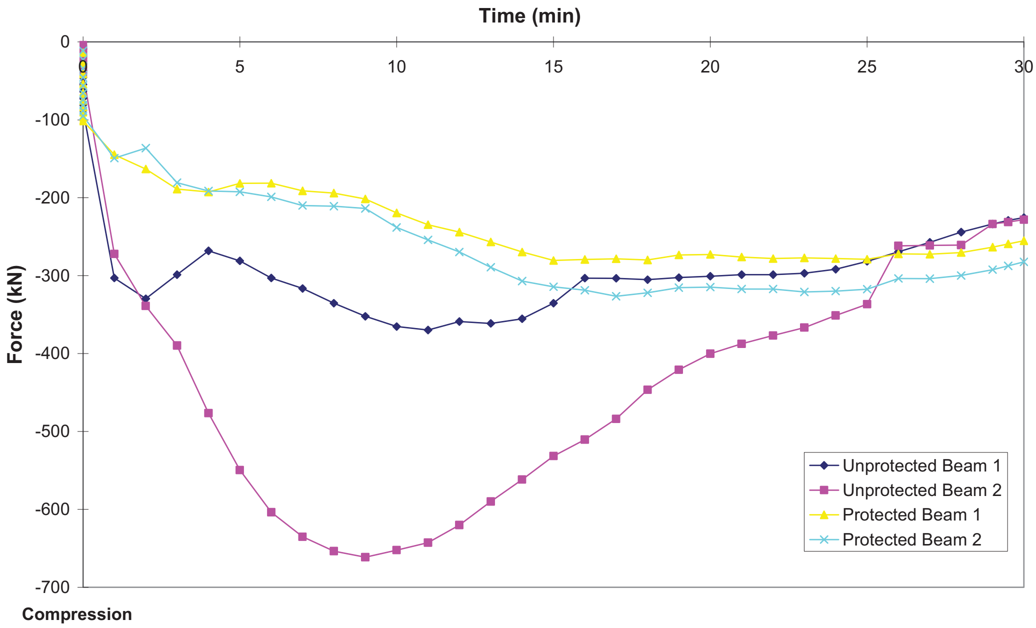


Fig. 26.5 Axial forces in typical connections

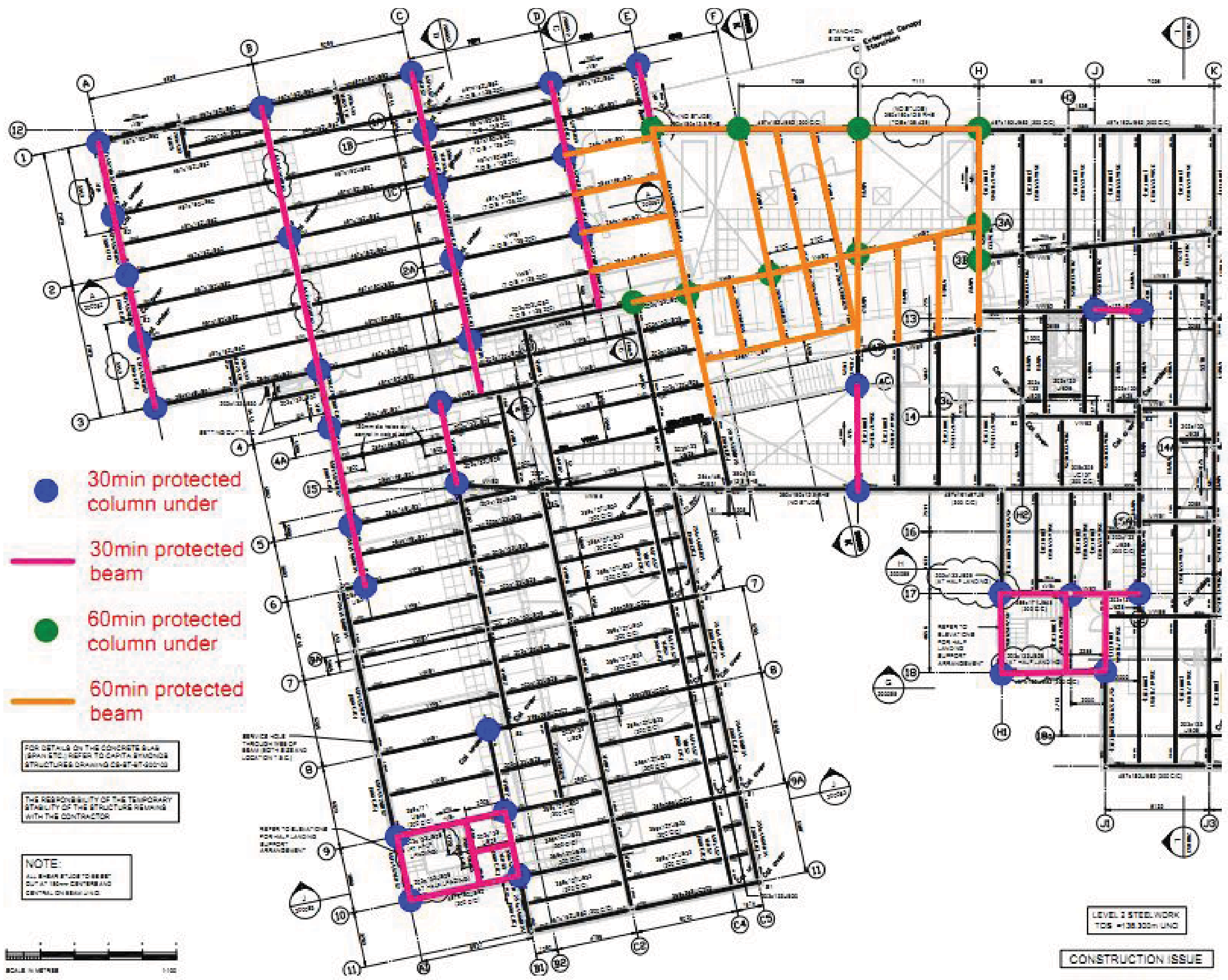


Fig. 26.6 Fire protection steelwork mark-up Level 2 (partial)

26.9 STAKEHOLDER APPROVAL

Before undertaking the fire engineered analyses it was necessary to hold meetings with the relevant stakeholders (Contractor, Building Control and Fire Service) both to explain the principles and intent and also to agree certain study parameters such as ventilation sensitivity studies for the time equivalence analysis. This method of gaining stakeholder approval prior to performing the analysis is in concurrence with the Qualitative Design Review (QDR) procedure as defined in BS 7974.

Due to Exova Warringtonfire's extensive experience in detailed fire engineered analysis no significant problems were encountered when seeking approval for the analysis. Questions received generally related to the ongoing maintenance requirements for the project once complete.

26.10 CONSEQUENCE ON LIFECYCLE OF BUILDING

The principle consequence on the lifecycle of the building relates to future use should a subsequent owner wish to change the principle compartmentation lines on a given floor. The recommendations are also contingent on a sprinkler system being maintained on the premises.

Additional important points for the contractor were as follows:

- Reinforcement mesh in the slab must be sufficiently lapped to form a full tension lap.
- All beams should be composite and the slabs should be tied to the beams.
- Connections to protected columns or beams should also be protected.
- Connections should be designed to be ductile.

As the use of the building will be a secondary school it was considered that the lifecycle of the building within this use group would be extensive and therefore compartmentation lines are unlikely to change during this period. However, the relevant stakeholders were made fully aware of this issue prior to undertaking the analysis and informed that any future change of use may be restricted should the current structural fire protection scheme wish to be maintained.

26.11 CONCLUSIONS

Fire engineering analysis has been carried out on the described building in order to demonstrate that the building will fulfill the functional requirement of the UK Building Regulations with regard to structural fire protection, with a reduced overall standard and coverage of said protection. The principle outcome for the project contractor was decreased costs in terms of the amount of intumescent paint required and the associated costs of transportation and application. The reduction in required fire protection is significant and may indicatively be seen in Figure 6 for one area of the building. The analyses were carried out with the full consent of relevant stakeholders and approvals authorities following a detailed QDR. The building is now complete, occupied and functioning as intended.

Acknowledgement

The authors would like to thank Bovis Lend Lease Ltd, Ellis Williams Architects, Carillion Plc, Lancashire Fire and Rescue Service and Andrew McCracken of Exova Warringtonfire for their contribution and assistance throughout the project.

References

The Building Regulations 2000: DCLG, TSO (The Stationery Office) *note: current at time of project*

Approved Document B, Volume 2 – Buildings other than dwelinghouses, 2006 Edition: TSO (The Stationery Office) ISBN 978-0-11-703725-0.

BS 476-20 1987: Fire tests on building materials and structures. Method for determination of the fire resistance of loadbearing elements of construction.

BS 7974 2001: Application of fire safety engineering principles to the design of buildings. Code of practice.

PD 7974-3 2003: Application of fire safety engineering principles to the design of buildings. Structural response and fire spread beyond the enclosure of origins (sub-system 3).

PD 7974-1 2003: Application of fire safety engineering principles to the design of buildings. Initiation and development of fire within the enclosure of origin (sub-system 1).

Eurocode 4, Design of composite steel and concrete structures, Part 1-2: General Rules Structural Fire Design: 2005.

BS 5950-8 2003: Structural use of steelwork in building. Code of practice for fire resistant design.

WG1- Florian Block, florian.block@burohappold.com
Susan Deeny, Tad-Song Kho and Gary Walker

27 STRUCTURAL FIRE ASSESSMENT OF THE ME HOTEL ALDWYCH, LONDON

Summary

This case study outlines a structural fire engineering review that has been carried out on the atrium steelwork within the ME Hotel project in London. Here a methodology to determine the amount of fire protection on the atrium steelwork is described, whilst meeting the requirements of the Part B of the English Building Regulations. In principle, the assessment was done in two steps; a hazard identification and risk assessment, and a structural analysis at elevated temperatures. For the worst fire scenario a heat transfer and thermal structural finite element analysis was carried out. An initial fire protection regime was proposed for the finite element study and is subsequently proven to be adequate in maintaining the global stability of structure. At ground floor level, where the frame will enclose a single storey compartment, the steelworks will be provided with 120-minute fire protection. From the first to tenth floor the steel frame encloses the atrium; the steelwork at the first floor level and the corner elements that run up the apexes will be protected to a 60-minute standard. The remainder of the atrium structure can be left unprotected.

27.1 INTRODUCTION

This case study describes the structural fire engineering assessment conducted on the ME Hotel project in London. The building consists of a mixed hotel-residential development. The residential apartments are located in the North East end of the building whereas the hotel accommodation is located in the remainder of the building. See Fig. 27.1 and Fig. 27.2 for plan and elevation of the building. In this case study only the hotel part of the building is looked into.

The main feature of the building is a steel truss structure, which is continuous from the Ground Floor to the uppermost hotel floor. Being triangular in plan, this structural frame gradually decreases in area with respect to height. The enclosed area within the triangular structure at ground floor level would be a restaurant; the structure from first floor onwards would be an open atrium space, to be used as a reception and check-in area. The atrium structural frame will be clad with a natural marble; this cladding forms a complete physical separation between the atrium and the hotel corridors which surround it.

This atrium is structurally crucial to the building by:

- Carrying gravity loads – Secondary beams carrying the floor slabs frame into the edge beams which form part of the atrium structure.

- Providing lateral stability – Works in conjunction with the concrete cores to the north-east and south-west of building.

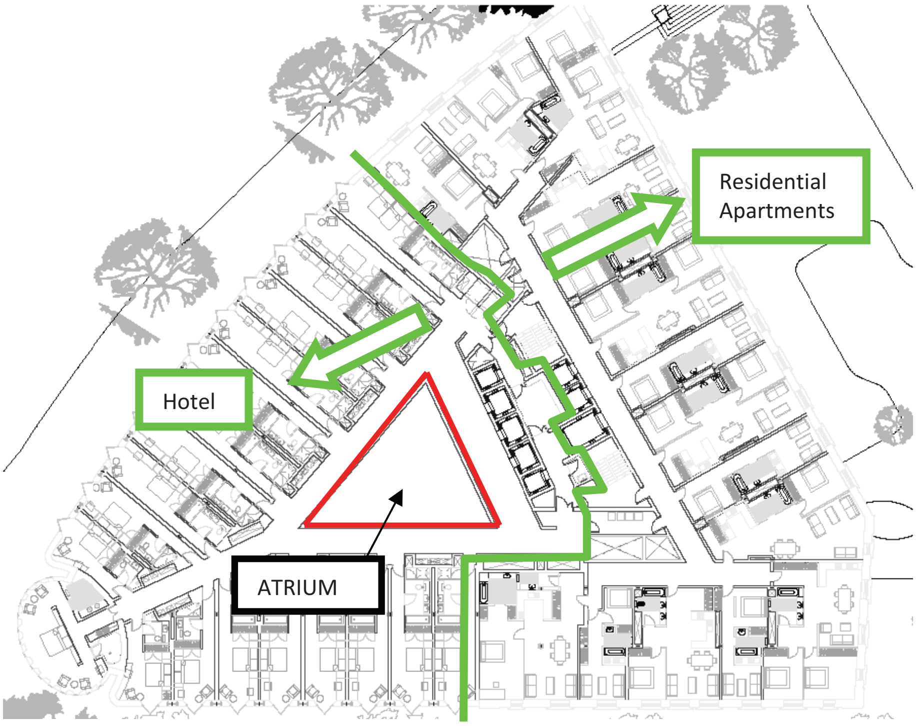


Fig. 27.1 – Typical floor plan of building

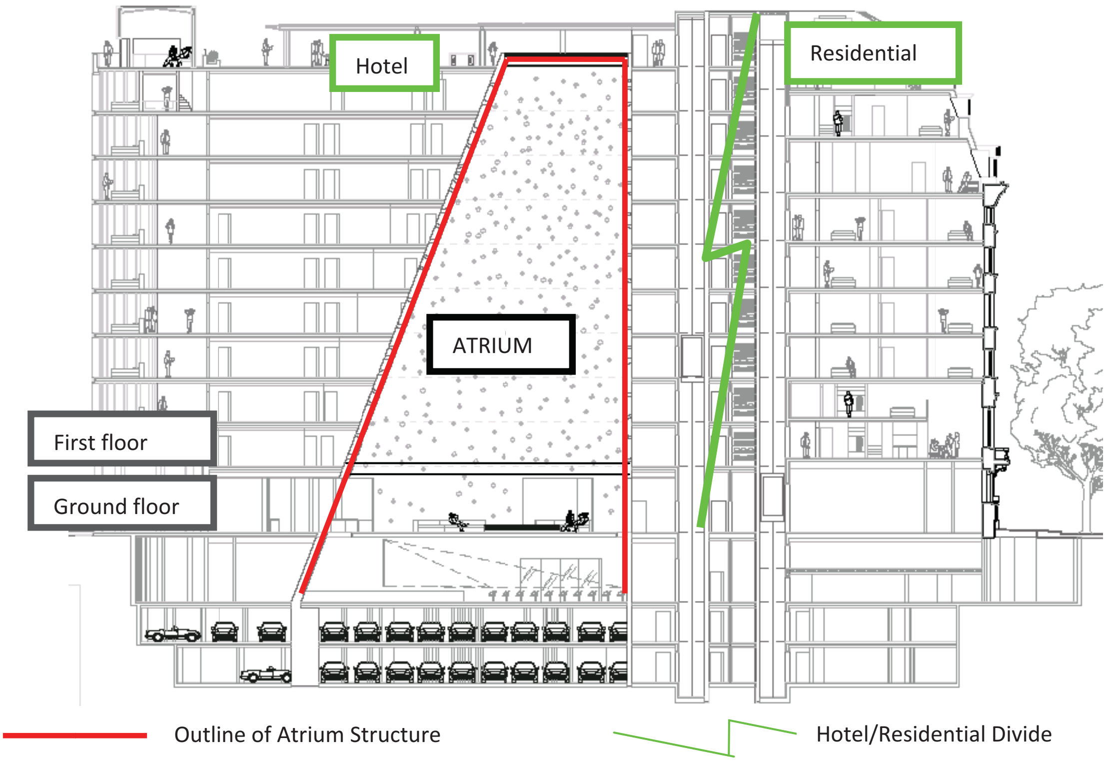


Fig. 27.2 Elevation of building

Prescriptively in accordance with Approved Document B (ADB), the atrium steelwork is required to achieve a 120-minute fire resistance period. However, within the atrium the remoteness of the steelwork from potential fire load, the protection of the stone cladding system and the provision on an automatic sprinkler protection system opens up the possibility of a performance-based design whereby only a limited amount of additional fire protection might be required for the atrium steelwork. This case study aims to address this.

27.2 STRUCTURAL FIRE ENGINEERING ASSESSMENT

A performance based design is adopted to determine the amount of fire protection needed for the atrium structure. Realistic fire scenarios and the risk they posed are considered based on the use of the building. The performance of the structure is then analysed considering thermal effects such as expansion and degradation of material strength and stiffness. The results are assessed against predefined acceptance criteria to ensure a safe solution.

27.2.1 Methodology

In principle, the study is carried out in three stages:

1. hazard identification and risk assessment
    - list all possible fire scenarios
    - undertake risk assessment and reduce risk if possible
2. thermal response modelling at elevated temperature
    - define design fire
    - calculate the heat transfer to the structure
3. structural response modelling at elevated temperature
    - calculate the response of the structure at the elevated temperature

For this case study emphasis is placed on the second and third stages.

27.2.2 Acceptance criteria

The acceptance criterion for the assessment is for the structure to maintain its global stability throughout the design fire duration. Local damage and failure of structural elements is therefore acceptable where this failure does not adversely affect the global stability of the structure. The rate of vertical deflection of the structure will be used as an indicator of stability. A rapid increase in vertical deflections is commonly associated with a loss of stability.

27.3 DESIGN FIRE

A hazard identification and risk assessment has been carried out and the worst realistic fire scenario is an unsprinklered fire at the atrium. The atrium base fire is used in the finite element analysis to determine the required fire protection scheme; as the more onerous design fire this will result in a conservative design for all other design fires. The base of the atrium space would be at first-floor level, where it is intended to be used as a reception and check-in area.

Due to the height of the atrium the fire cannot be controlled with sprinklers. A fuel load restriction is imposed on the atrium base to ensure the peak heat release rate is no more than 2.5MW for the purpose of the atrium smoke control system. Below are the details of the fuel load to be used:

- Fire loads should be placed in islands containing a maximum of 185kg of combustible material,
- The area of any one fire load island should not exceed 10m^2, and
- The islands should be separated by a minimum distance of 3m.

The design fire was modelled as αt^2 squared growing fire of moderate growth rate. For the purpose of the structural assessment the heat release rate has been capped at 2.7MW and no decay phase has been considered to provide a degree of conservatism. This has been based on experimental data provided in the BRE Design Fire Guide (2002). For the growth rate and heat release rate defined the total time taken to consume the available fuel within an island is 30.5 minutes. The structural elements that need to be protected are therefore provided with 60-minute fire rated protection. The performance of the structure was analysed for the full duration of the fire resisting rating. Figure x.3 shows the design fire heat release rate.

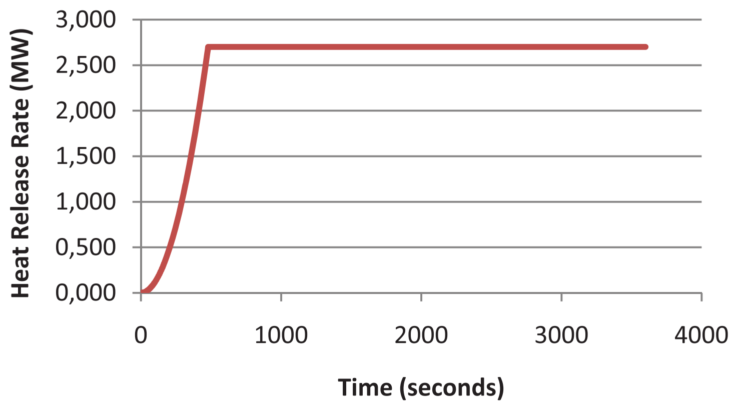


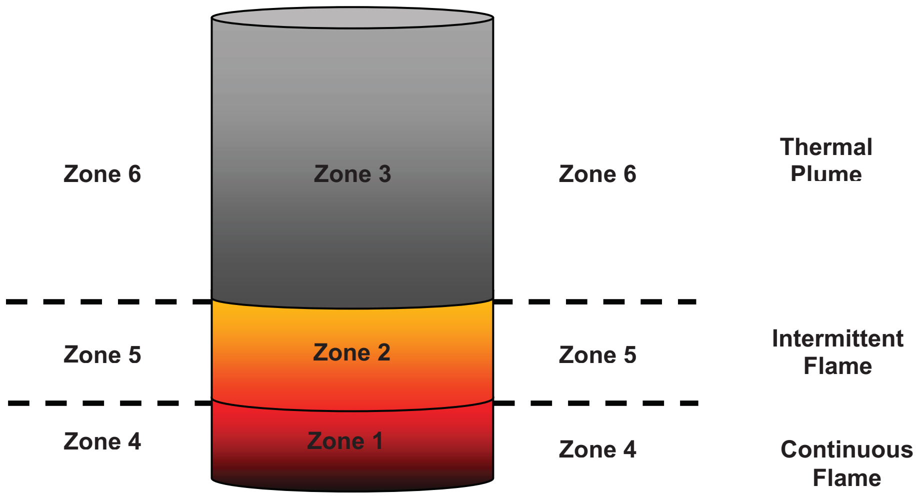
Fig. 27.3 Design fire heat release rate

27.4 THERMAL RESPONSE MODELLING

27.4.1 Heat transfer analysis approach

The large volume of the atrium, limited fuel load and provision of smoke ventilation will prevent the smoke layer from becoming hot enough to lead to flashover. The fire therefore remains localised with high gas temperatures also localised to the fire base. The thermal exposure of structural elements remote from the fire is therefore dominated by radiation.

A localised fire heat transfer model which represents the fire as a layered cylinder of different temperatures was therefore employed. For this scenario the cylinder is composed of three temperature layers based on the geometric components of a localised fire: the continuous flame, the intermittent flame and the thermal plume. The atrium space is then divided into 6 zones based on the three geometric components of the fire as illustrated in figure 27.4



Zone 1	Within the continuous flame – A cylinder of radius equal to that of the base of the flame
Zone 2	Within the intermittent flame – A cylinder of radius equal to that of the base of the flame
Zone 3	Thermal Plume above the flame – A cylinder of radius equal to that of the base of the flame
Zone 4	Adjacent to the Continuous Flame – At the same elevation as the continuous flame but at a radius greater than the base of the flame
Zone 5	Adjacent to the intermittent Flame - At the same elevation as the Intermittent flame but at a radius greater than the base of the flame
Zone 6	Above and Adjacent to the flame - At an elevation above the flame and at a radius greater than the base of the flame.

Fig. 27.4 Cylinder model for heat transfer analysis

Using the cylinder model described above it is permissible to calculate the net heat flux from which protected and unprotected steel temperatures are calculated using the lumped mass approach of Eurocode 3 Part 1.2 (Section 4.2.5.1). Intumescent paint is to be used to provide fire protection. For the protected members artificially increased densities have been used to reduce the rate of temperature increase mimicking the insulating effect of the intumescent paint. This is referred to here as pseudo density.

The appropriate pseudo density was calculated using a goal seeking approach, where the goal is set as the assumed steel section failure temperature (620ºC) at the prescribed fire resistance rating of the applied intumescent paint (FR60). It has further been assumed that after reaching the fire resistance period the fire protection will fall off completely and therefore the element will be treated as unprotected thereafter. Figure 27.5 contrasts the standard temperature time curve and temperature evolution for an unprotected and intumescent 60 min protected element.

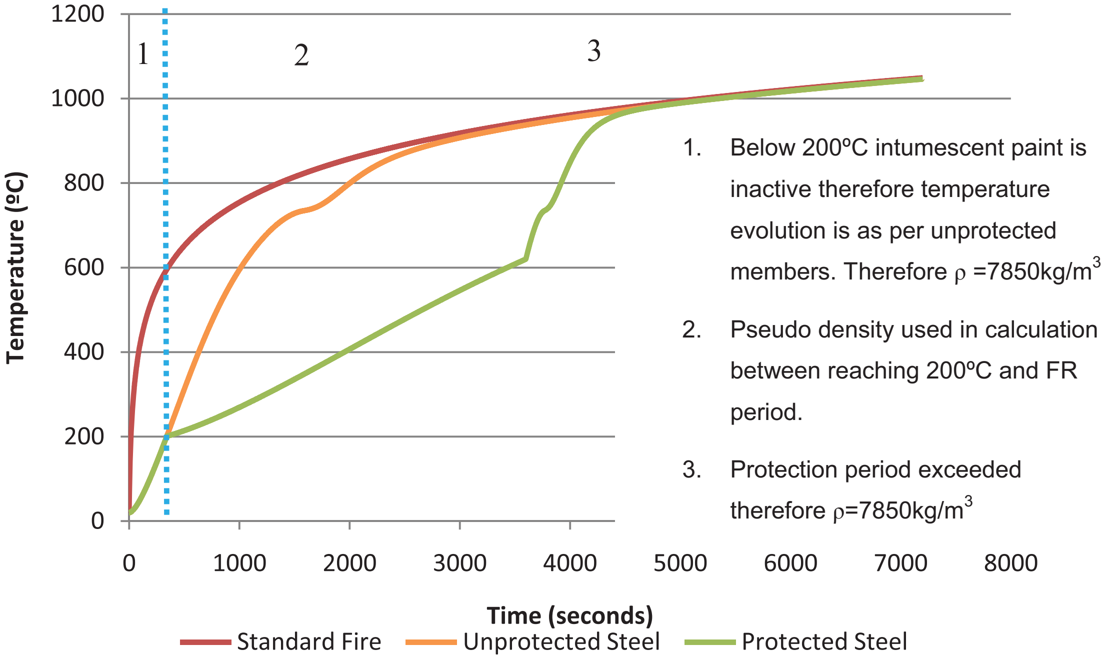


Fig. 27.5 Protected and Unprotected steel temperature evolution

27.4.2 Assessed fire scenarios

It has been identified that the vertical corner elements (legs) of the frame are critical elements for frame stability. The worst case location of the fire on the atrium base is therefore in close proximity to a leg. Symmetry allows us to consider just two fire locations. The performance of the atrium frame is analysed under four different fire and loading combinations. These are summarised along with the indicated performance result in Table 27.1.

Tab. 27.1 Analysis Combinations

Fire\ Loading	Dead + Live	Dead + Live + Wind
Nose	Maintains Global Stability	Maintains Global Stability
Rear North	Maintains Global Stability	Maintains Global Stability

27.4.3 Determination of fire protection scheme

The determination of the required protection scheme was conducted in an iterative fashion. From interrogation of the frame to identify key elements for stability the following fire protection scheme is proposed as a first estimate:

- atrium steelwork between first floor level and second floor level the protection is applied as outlined in Fig. 27.6 – these elements are likely to receive large amounts of radiation in the event of a fire

- column at apexes from second floor onwards - 60 minutes fire protection – these elements have been identified as critical for the structural stability of the frame

If the analysis results do not achieve the acceptance criterion (mentioned in Section 27.3.2) the frame is assessed for points of weakness by the user. The analysis is then re-run providing protection to this area.

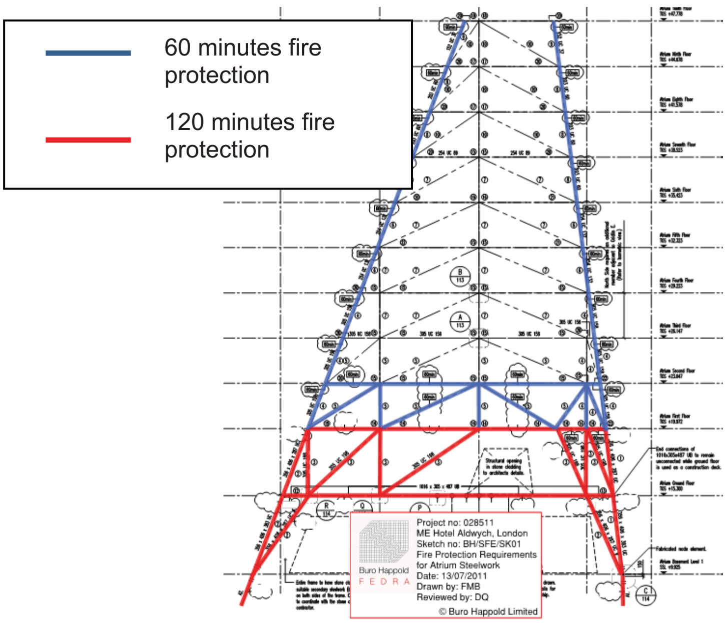


Fig. 27.6 – Fire protection layout on atrium structure

27.4.4 Heat transfer to structural members

The analysis of the thermal response of the structural frame does not include the beneficial shielding or insulating effects by the proposed marble cladding system mentioned in Section 27.2.3 as it was not possible to guarantee performance of the stone cladding in the areas where the direct flame impingement could be possible.

The thermal response of each individual structural element has been calculated using the methodology described in Section 27.6 for the nose and rear north corner design fires. This calculation considered the effect of the intumescent paint 60 minute fire protection applied to the steelwork stated in Section 27.4.3. Figure 27.7 illustrates the atrium frame temperature profile after 60 minutes exposure for a fire located in the (a) nose of the atrium and the (b) rear north corner. The majority of the atrium frame remains quite cool (< 100ºC) for the duration of exposure. The figure caps the temperature contours at 220ºC to illustrate the extent of area where the intumescent paint if present is activated; these are depicted in grey.

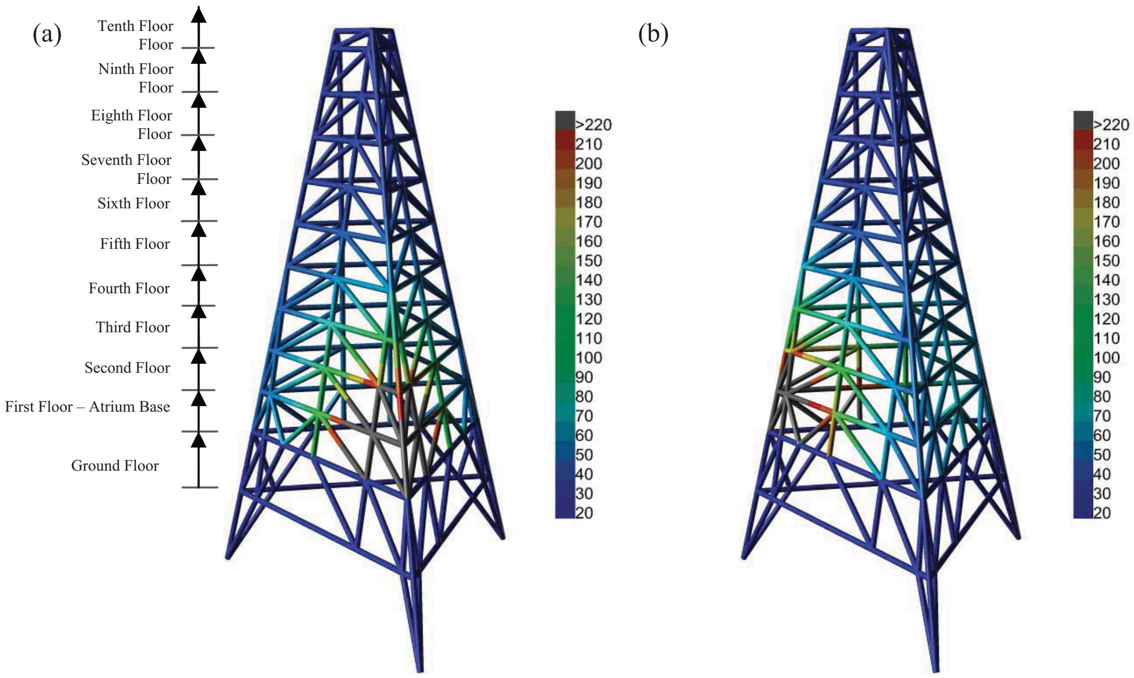


Fig. 27.7 Structural Frame temperature profile 220ºC cap (ºC) after 60minutes (a) Nose Fire (b) Rear North Fire

27.5 STRUCTURAL RESPONSE MODELLING

27.5.1 Structural analysis methodology

The structural analysis was conducted using the finite element (FE) software VULCAN, a special purpose high-temperature structural analysis FE package. The acceptance criterion for this assessment is for the structure to maintain overall stability for the duration of the previously defined design fire. Local damage and failure of structural elements is therefore acceptable where this failure does not adversely affect the global stability of the structure. The rate of vertical deflection of the structural elements within the frame will be used as an indicator of stability; a rapid increase in vertical deflections is commonly associated with a loss of stability.

27.5.2 Description of Vulcan model

Vulcan was used to carry out the finite element analysis, using mainly beam elements. The geometry of the structure was been obtained from the structural engineers and imported into Vulcan.

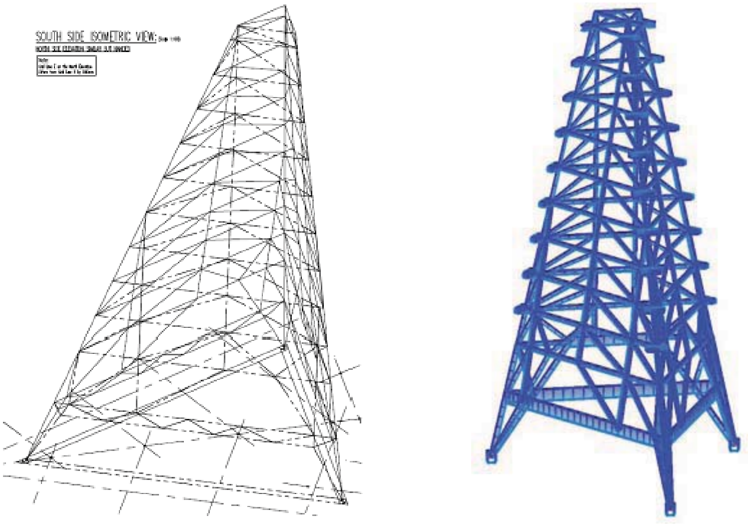


Fig. 27.8 Structural Frame (a) Isometric view (b) Vulcan view

27.5.3 Loading

For the structural analysis, load factors at fire limit states used for the design are as stipulated in BS5950 Part 8, which are 0.8 for imposed loads, 1.0 for dead loads and 0.33 for wind loads.

27.5.4 Modelling effects of restraints

The vertical component of the base support is modelled as fixed, that is infinitely stiff. The lateral components are restrained using directional springs.

The atrium frame is restrained by the surrounding structure at each level; thermal expansion is known to have both beneficial and detrimental effects it is therefore necessary to include the effects of this restraint within the VULCAN model.

Firstly, the floor slabs of the surrounding structure are tied to the horizontal frame members as indicated in Figure 27.9(a). The restraining effect of the floor slabs for the horizontal members has been represented by the inclusion of small width (1m) slab elements which are tied to the perimeter beams. These slab elements are non-load bearing, their effect upon the model was limited to the provision of longitudinal restraint of the perimeter beams. The location of these slab elements is illustrated in Figure 27.9(b), where the slab elements are depicted in red. Slab elements were only applied from second floor onwards, as the steelwork from first floor downwards are not going to be affected by the fire at the base of the atrium.

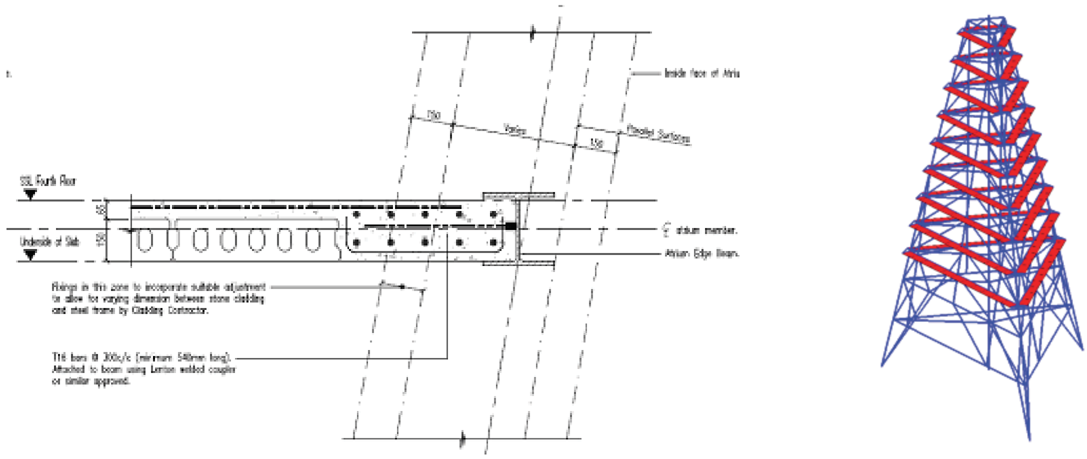


Fig. 27.9 (a) Edge beam to floor detail (b) Slab panels providing lateral restraint

Secondly, the atrium frame is a component of the buildings lateral stability system. The building has been designed to very low lateral movement tolerances (height/1000) to minimise any damage to the retained historic façade. The building's lateral stability system comprises three elements, a curved concrete wall on the western façade, the atrium steel frame and a central concrete core constituted by two shear walls. The location of these on a typical floor layout is illustrated in Fig. 27.10(a).

To gain a realistic assessment of the performance of the structure in the event of a fire to include the lateral restraint imposed by the concrete west wall and central core upon the atrium. This restraint to the atrium frame has been modelled as a stiff column with a fixed base tied to the atrium floor at each floor level. In Figure 27.10(b) the location of the column and the sections that tie it to the atrium frame are depicted in red. The bending stiffness at each level has been deduced from the atrium frame structural calculations.

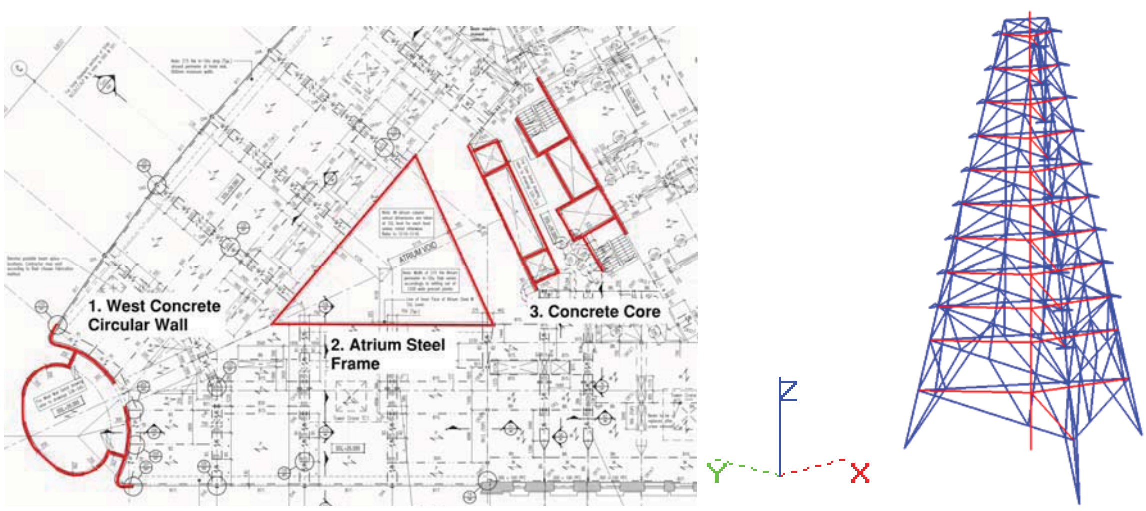


Fig. 27.10 (a) Location of core walls (b) VULCAN Model Core Representation

27.5.5 Structural response

The vertical deflections at the three corners of the 10th floor level were used to check the rate of deflection during the fire exposure; these locations are illustrated in Fig. 27.11.

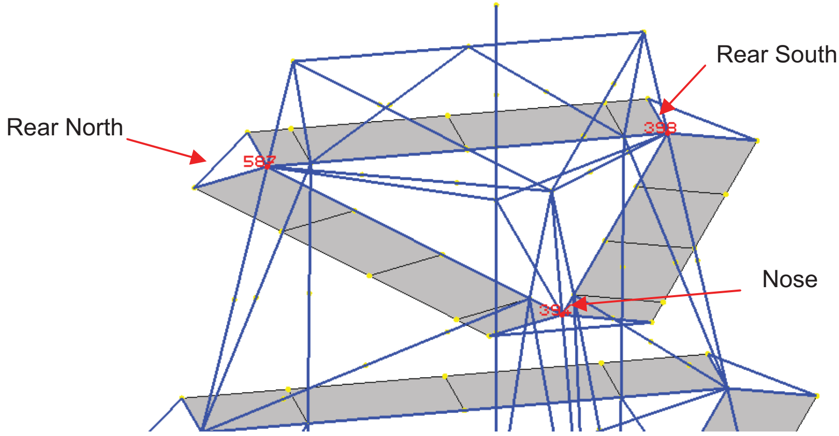


Fig. 27.11 10th floor locations of vertical deflection measurement

<u>Rear North Corner Fire</u>

Figure 27.12 graph the evolution of the 10th floor vertical displacement for the duration of exposure for the rear north corner fire under dead, live & wind loading respectively. The vertical deflection of the atrium frame under a fire located in the rear north corner is characterised by a relatively quick increase in deflections from about 5 to 15 minutes for both load combinations. Quickly increasing deflections in the early stages of exposure are related to the rapid heating of structural elements local to the fire to 200ºC whereupon the intumescent fire protection activates. During this time some buckling of isolated members at first floor level is evident; however as for the nose located fire the subsequent decrease in deflection rate indicates that this buckling does not adversely impact upon global stability.

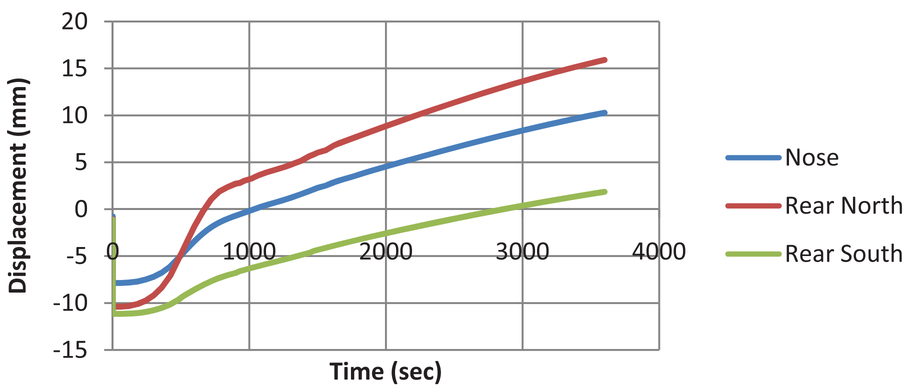


Fig. 27.12 Rear North corner under dead, live & wind loading – 10th floor vertical displacements

Deformation plot

Figure 27.13 shows the deformation and local buckling occurring to the structure of the structure at 60 minutes for a dead and live load combination.

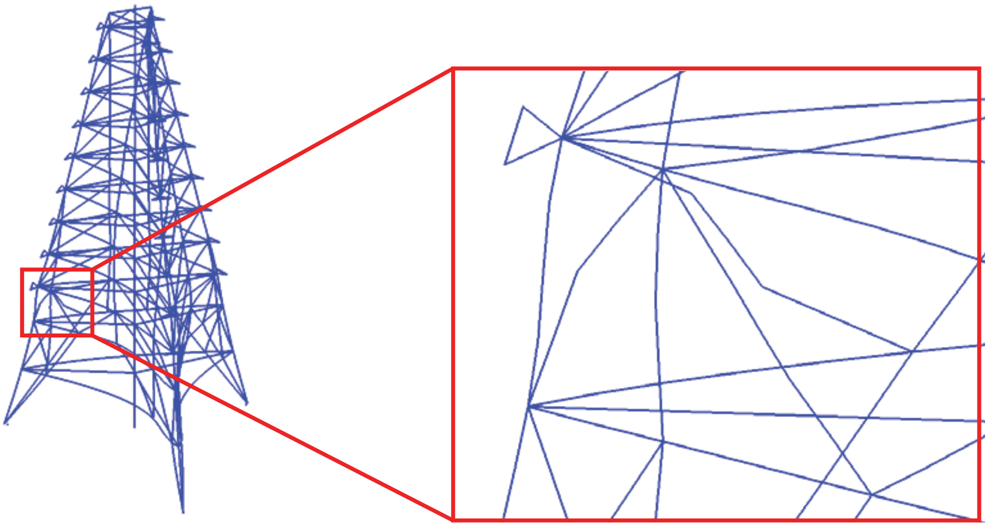


Fig. 27.13 Deformation plot for nose fire scaled at 35.

27.5.6 Discussion of results

The results of these analyses indicate that the structure maintains global stability during exposure to a localised fire subjected to the thermal analysis and structural analysis described under Section x.4 and Section x.5 respectively. This implies that the preliminary fire protection scheme stated in Section x.4.3 is sufficient in ensuring the load carrying capability of the atrium structure in an event of a fire, despite some buckling of steel members at the vicinity of the fire.

27.6 POST-STUDY

A report based on the findings in this study was submitted to the local building authority and has been subsequently approved. The fire protection scheme, as indicated in Fig. 27.6 was communicated to the steel fabricator, whereby the amount of fire protection needed to achieve the required fire rating is calculated in accordance to BS476.

27.7 SITE IMAGES

As this case study was written, construction on the building was in its final stages. The proposed fire protection regime outlined above was followed, with the exception of the steelwork between first floor level and second floor level, in which all members were fire protected. Below are some pictures of the atrium steelwork.

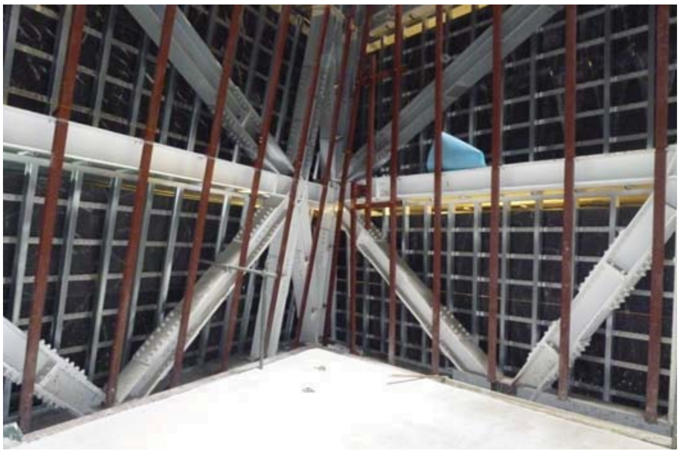


Fig. 27.14 Intumescent paint sprayed on site at first floor

27.8 CONCLUSIONS

The paper summarises a structural fire assessment carried out on an atrium structure. While the prescriptive guide recommends the whole structure to be protected by a 120-minute fire ratings, engineering judgement suggests that the structure could be partially protected without compromising the stability of the structure in an event of a fire. A hazard identification and risk assessment was carried out on the possible fire scenarios. After identifying an unsprinklered fire at the atrium base as the worst case and assuming a preliminary fire protection scheme, a heat transfer analysis was then conducted. The results of the thermal analysis implies that compartmentation on two floors could be breached therefore a 60-minute compartment walls are proposed on those floors. Considering thermal effects, the structure was then analysed in Vulcan subject to different fire positions and different load combinations. The predicted structural response indicates that the global stability of the structure is maintained throughout the duration of the fire period. Thus it has been demonstrated that only certain members require fire protection (60 minutes within the atrium or 120 minutes out with the atrium) while majority of the steelwork within the atrium can be left unprotected. This case study demonstrates that combined rationalisation of the potential fire scenarios and consideration of global structural performance can produce an optimised structural fire protection scheme which meets the life safety objectives of the Building Regulations.

Acknowledgement

The author would like to thank Meliá Hotels International and Fosters & Partners for their permission to present this work. Meliá Hotels International is the hotel developer based in Palma de Mallorca, Spain. Information about the company can be found via MELIA.COM, facebook.com/MeliaHotelsInternational or twitter.com/MeliaHotelsInt.

<u>References</u>

BRE, Design Fires Database 2002, BREPress.

BS 7974-1: 2003 Application of fire safety engineering principles to the design of buildings — Part 1: Initiation and development of fire within the enclosure of origin (Sub-system 1). BSI.

BS 5950-8:2003 Structural use of steelwork in building – Part 8: Code of practice for fire resistant design. BSI.

BS EN 1991-1-2:2002 Eurocode 1: Actions on structures — Part 1-2: General actions — Actions on structures exposed to fire. BSI.

BS EN 1993-1-2:2005 Eurocode 3: Design of steel structures — Part 1-2: General rules — Structural fire design. BSI.

Department of the Environment, Transport and Regions and Welsh Office, The Building Regulations - Part B, 2000, HMSO.

London Building Acts (amendment) Act 1939 (as amended by the Building (Inner London) Regulations 1985). Section 20. {Fire Safety Guide No.1. Fire Safety in Section 20 Buildings. London District Surveyors Association. 1997}.

Office of the Deputy Prime Minister, 2000. Approved Document B - Fire safety - 2000 edition consolidated with 2000 and 2002, TSO (The Stationary Office), Norwich.