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Contract RFS-04048 "Stainless Steel in Fire"

WP 6 Final report

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1. EXECUTIVE SUMMARY

It is well known that stainless steel has excellent fire performances and then could be a very interesting solution for building applications where carbon steel would require fire protection. To investigate potential applications, numerical studies have been carried out on fire behaviour of both unprotected stainless steel members under natural fire conditions, such as columns in open car parks, and stainless steel members located outside the buildings. From obtained results, design rules for external stainless steel columns in fire as well as those in open car parks have been proposed.

CONTENTS

2. INTRODUCTION

The purpose of the present research work was to illustrate that stainless steel can become a practical alternative solution to protected structural carbon steel. So, different applications were investigated in which it is possible to use unprotected stainless steel, whereas carbon steel in the same application would require fire protection, such as external members or members submitted to natural fire conditions. More exactly, it was selected to investigate:

- the fire resistance of exposed stainless steel columns located outside buildings using a numerical modelling with help of the computer code ANSYS which was proved already to be in good agreement with several fire tests performed on stainless steel members [1]. Moreover, heating of external columns has been calculated from simple calculation methods developed in one previous ECSC Project "*Development of design rules for the fire behaviour of external steel structures"* (contract number 7210-PR-380) [2].
- the fire behaviour of unprotected stainless steel columns in open car parks. This study has been performed on different framing systems (with unprotected carbon steel and concrete composite members) commonly used in France from a fire safety engineering procedure developed at CTICM and validated against experimental results [3, 9].

From the results of these numerical studies, fire design guidance for both external columns as well as those in open car parks has been developed. The proposed design method for external stainless steel columns allows to calculate the ultimate buckling resistance using the appropriate buckling curve with the 0.2% proof characteristic strength. It has been shown through comparison with numerical results that the proposed calculation methods give an adequately safe prediction of the fire resistance of external columns. Design guidance established for stainless steel columns in open car parks gives the maximum load ratio of steel column to be used for standard framing systems of open car parks.

The actual report describes numerical analyses, corresponding results and design guidance developed for investigated stainless steel members.

3. EXTERNAL STRUCTURES

3.1 NUMERICAL ANALYSIS

The fire behaviour of external stainless steel columns has been investigated using the computer package ANSYS for the current building configuration presented in Figure 1. It was supposed that the column was subjected to external flames from a fire compartment located at an intermediate level or at the top level of the building.

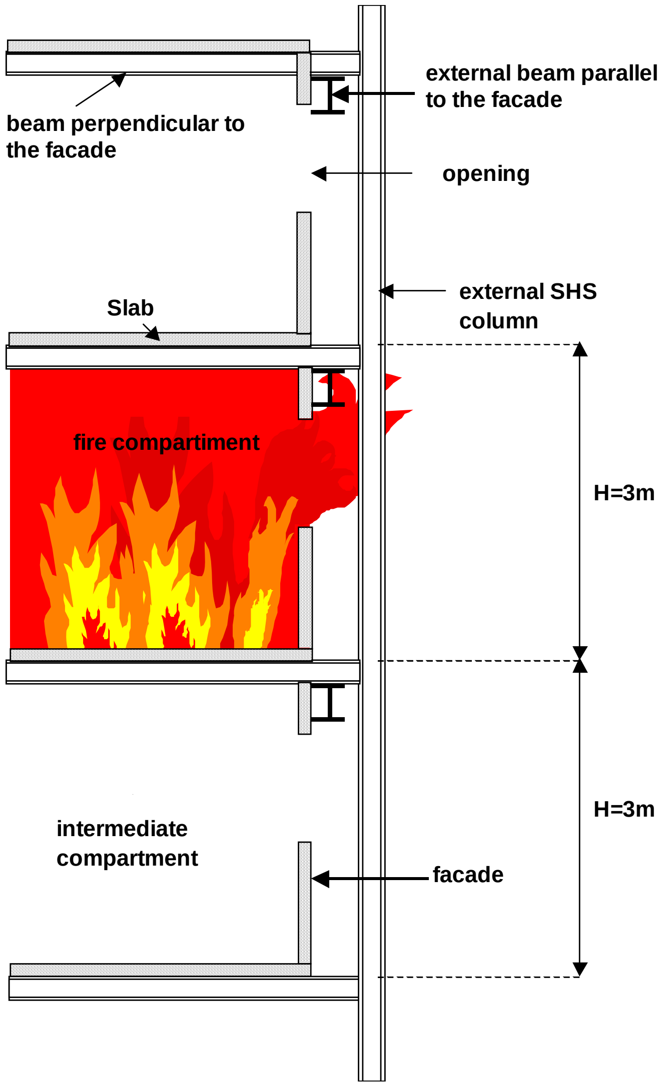


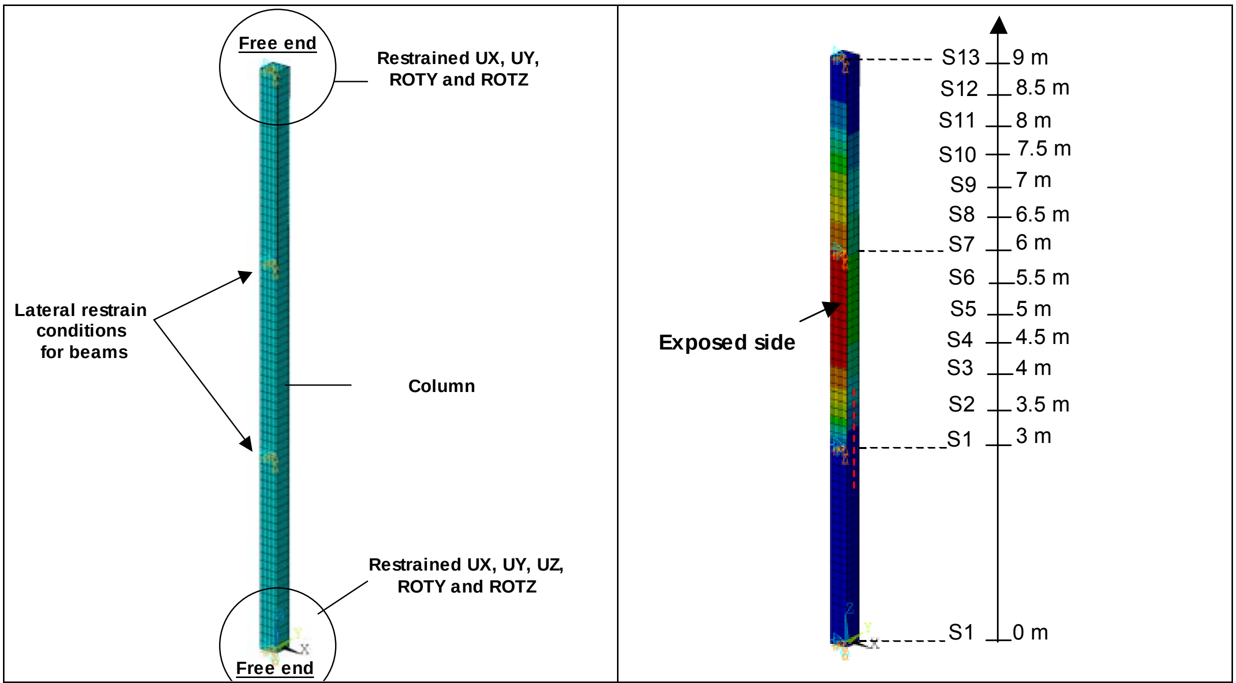
Figure 1 : Details of investigated external columns

3.1.1 Numerical assumptions

Numerical simulations have been performed using the following assumptions:

- Stainless steel column was modelled with non-linear beam-column element BEAM188. Column was meshed over a length of 9 m only (corresponding to the total height of 3 successive levels) to take into account the favourable effect of column continuity on its fire performance. Beams were not modelled. Only connections of column with horizontal steel beams were represented by appropriate boundary conditions restraining lateral displacements (restraining UX and UY) and rotation (ROTY and ROTZ) of column at the position of beams. Column ends were assumed to be hinged, so restraints were also applied to model the supports. Vertical load was applied on the column top. This load as its application direction (parallel to Z axis) was kept unchanged during the fire exposure (see Figure 2). An out-of–straightness of L/1000 was introduced in the modelling for geometrical imperfections.

- The mechanical material properties of the column as a function of temperature have been taken to be in accordance with EN 1993-1-2, assuming the following mechanical properties at room temperature: f_y = 240 N/mm², f_u = 530 N/mm², E = 200,000 N/mm². Poisson ratio was taken as a constant value of 0.3.



a) modelled structure	b) Temperature distribution along the column

Figure 2: Detail of numerical modelling of column

- A temperature gradient was introduced along the column length. This gradient was defined from temperature fields related to several cross-sections, assuming a linear variation of temperatures between these sections (see Figure 2). Each section was divided in four parts according to the column's surface. The temperature field was then described by the evolution of three appropriate temperatures: the temperature of the exposed side, the temperature of the unexposed side and the temperature of the two intermediate sides.

In our study, the heating of external stainless steel members were calculated using 2D simple calculation model developed in another ECSC Project "*Development of design rules for the fire behaviour of external steel structures"* (contract number 7210-PR-380) which was proved already to give a sound estimation of transient heat fluxes to carbon external members during a fire. This model allowing only the prediction of thermal actions to external structure involves following three stages:

- a zone model for the compartment fire,
- an external flames model,
- and the thermal actions calculation.

For the first stage, any zone model capable of simulating fully developed fire can be used. Once temperature and heat flows through the opening are known, the second stage consists in estimating the properties of external flames by using the GER concept. The third stage is then devoted to the calculation of heat flux to structural member, using view factor for non engulfed member for example. To take account of bowing effect due to temperature gradient in mechanical analysis, a simplified 2D heat transfer model has been also developed. This model allows estimating the temperature development on rectangular hollow steel columns located outside a building when external thermal actions are known. As the focus is laid on steel section, the assumption of a thermally thin solid has been made in modelling. The hollow section is divided into four parts corresponding to its four sides, assuming a uniform temperature within each part. The heat balance of each part takes into account heat exchange with exterior (mainly thermal action from fire) as well as radiative balance between the four parts within the cavity. Heat conduction is neglected between the four sides of the section and along the column. Using heat fluxes as source data, the output data of this simplified 2D heat transfer are then the temperatures versus time of the four sides of the considered section. The combination of the above models makes it possible to get a quite good prediction of the thermal behaviour of hollow steel sections located outside a compartment during a fire.

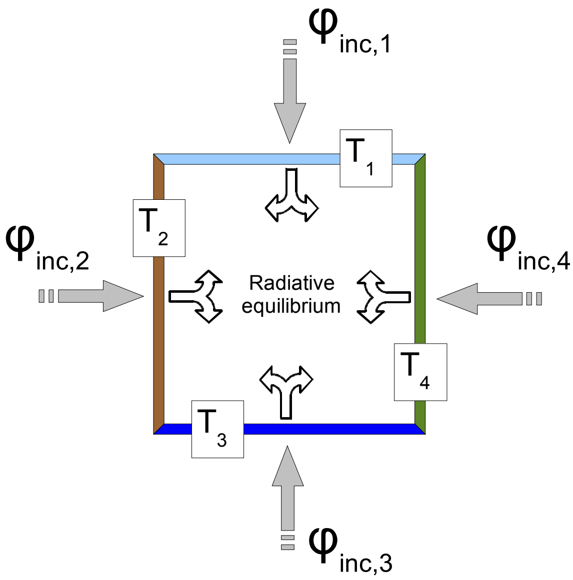


Figure 3: Principle of simplified 2D heat transfer model for hollow steel section

It should be noticed that simple methods developed for thermal analysis have been validated in case of stainless steel members against results given by advanced numerical models. A validation case study was proposed, on the basis of an experimental test performed by CTICM on external steel column exposed to a compartment natural fire. This test was carried out on a steel assembly composed of insulated beam (5m long) in a fire room and a steel column (4m long) located outside of the compartment (at 545 mm from the façade of external wall) but in front of the opening of the room. From this configuration, heating of one external square hollow steel column (300x8mm) has been calculated with previous models and compared to the temperature distribution obtained for the same column by a Computational Fluid Dynamics analysis (allowing to determine thermal actions) combined with a heat transfer analysis using advanced calculation model. In our study, the computer package ANSYS was employed.

Structural details of the case study are reported in the following figure.

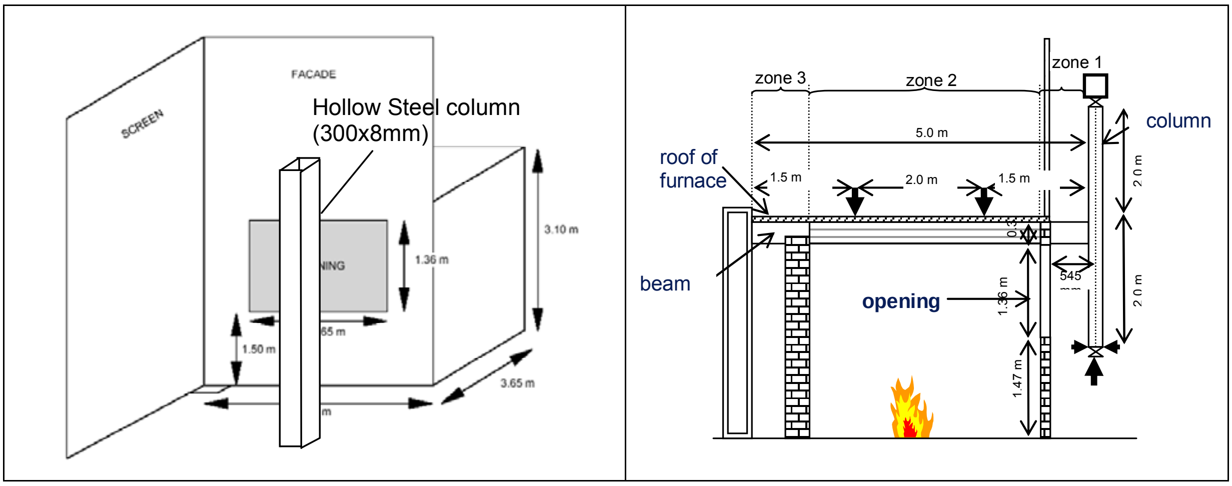


Figure 4: Details of case study

The temperatures on the hollow steel column have been calculated at level of seven sections (see **Figure 5)**. For each section, four temperature rises (one per face) have been predicted.

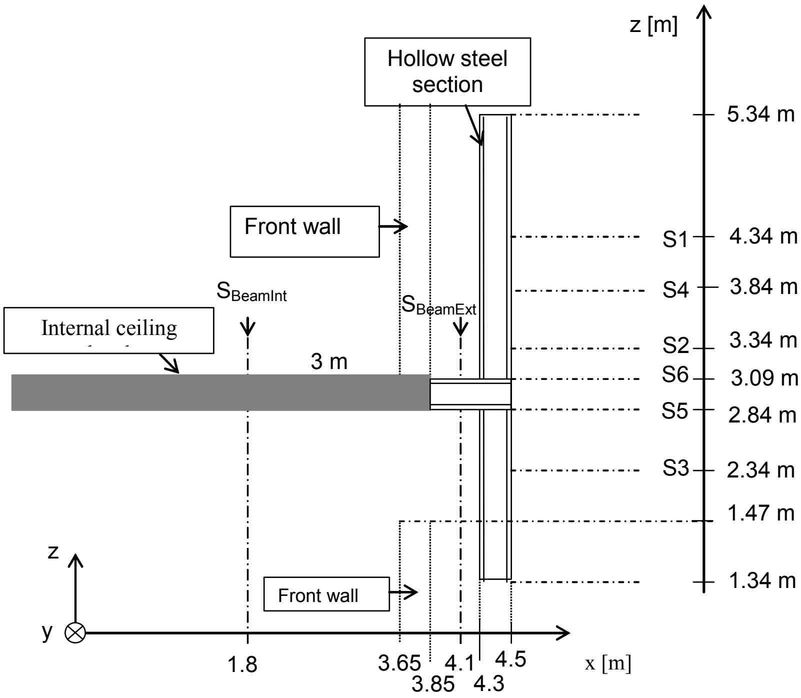


Figure 5: Sections distributions

As example, the following figure presents the temperature rises predicted in the steel column section S5. These temperatures are compared to the temperature distribution obtained for the same column by detailed numerical analyse (FDS + ANSYS). It can be noted that temperatures obtained from simple models are in good agreement with advanced numerical simulations.

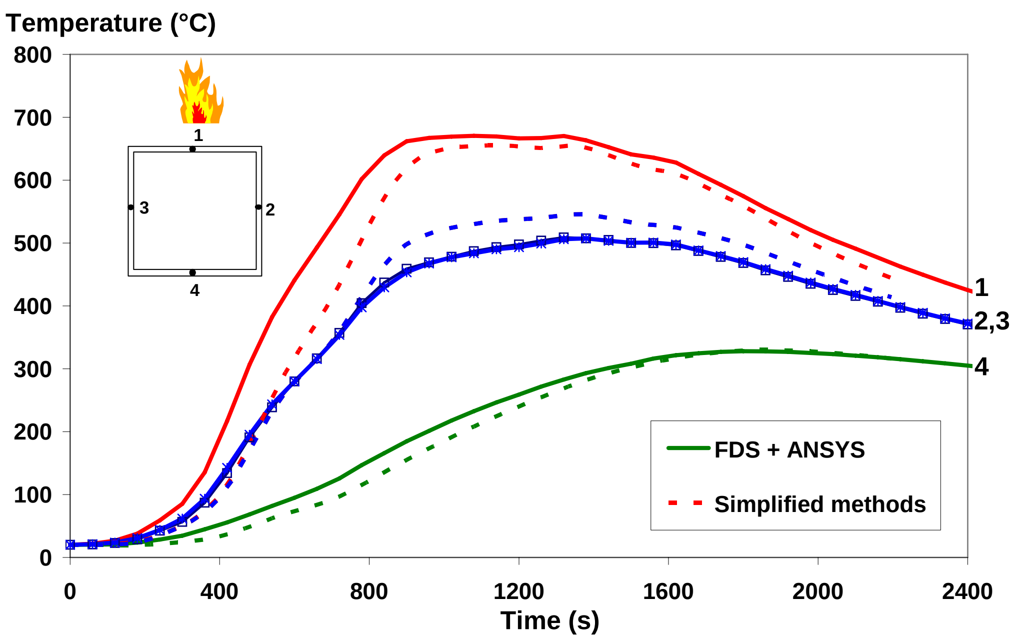


Figure 6: Temperatures rises in hollow steel column

3.1.2 Investigated parameters

Mechanical behaviour of external stainless steel columns has been assessed by varying the main parameters affecting the fire severity, the fire performance of columns. More exactly, main parameters adopted in parametric study were the following:

- <u>Compartment sizes</u>: 3 sizes, namely 3m×4m , 6m×4m and 9m×8m;

- Wall thermal properties: Density ρ=1000 kg/m^3, specific heat C=800J/KgK, thermal conductivity λ=0.5 W/mK, wall emissivity ε_w=0.9, wall thickness e_w=0.2m;
- External member location: 2 locations in front of the opening: 0.5m (engulfed in fire) and 1.5m from the compartment;
- Total area of windows: 25 %, 50 % and 100% of the façade area;
- Fire load density: 500, 750, 1000 and 1500 MJ/m²;
- External members: Square hollow section columns with cross-section ranging form 100 to 300mm and stainless steel grade EN1.4401;
- Load level of column: ranging from 0.1 to 0.7 (the load level is based on the design resistance of the members according to EN 1993-1-1 design rules);

3.1.3 Summary of numerical results

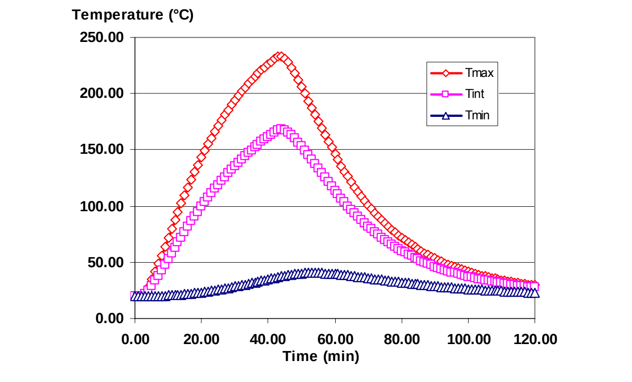
The results of this parametric study are quite numerous. Nevertheless, some representative results are given hereafter.

As example, Figure 7 show temperature developments calculated for the SHS column 150x8mm engulfed in fire assuming a compartment of 3m width and 4.0 m, a fire load of 1500MJ/m² and an opening of 50%. It illustrates very well one of the main features of the heating conditions of external steel members, namely strong thermal gradients along and across the section. For example:

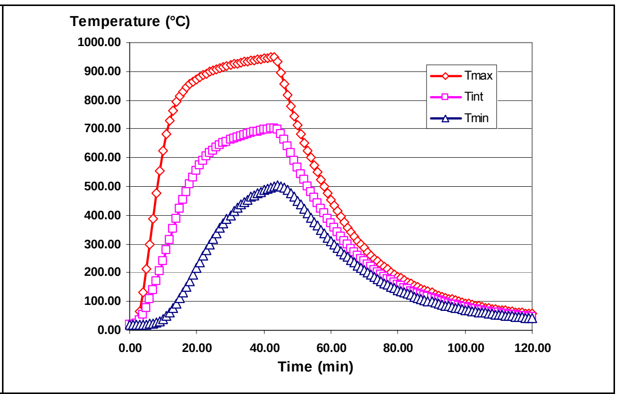
- Across the section of the column, temperature differences can easily reach 400°C to 500°C with reference temperature of 1000°C between both exposed and unexposed sides.
- Along the column, thermal gradient over the height can be about 500°C/m.

The most severe cross-sectional temperatures obtained at failure of some investigated cases are given in Table 1 It can be seen from this table that stainless steel columns have a mean temperature higher than 550°C at failure. Obviously the column temperatures at failure increases with lower load levels. Moreover, it is clear that the use of unprotected external stainless steel columns is fully possible if the load level is lower than 0.5.

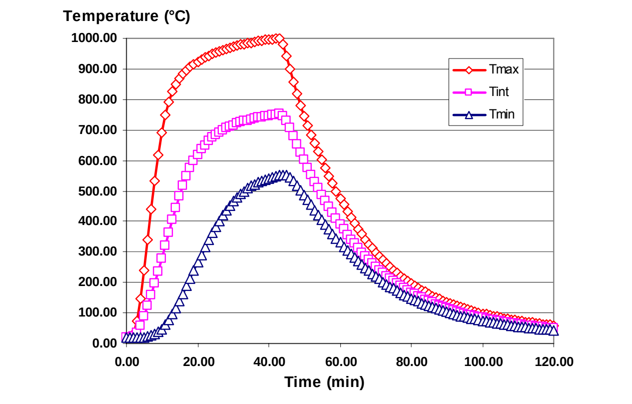
To confirm advantages of external columns in stainless steel with regard to the fire resistance, additional calculations have been made with S235 carbon steel column without any applied fire protection (see Table 2). It can be seen that carbon steel columns already failed before 30 minutes where stainless steel column can remain stable during whole fire duration.



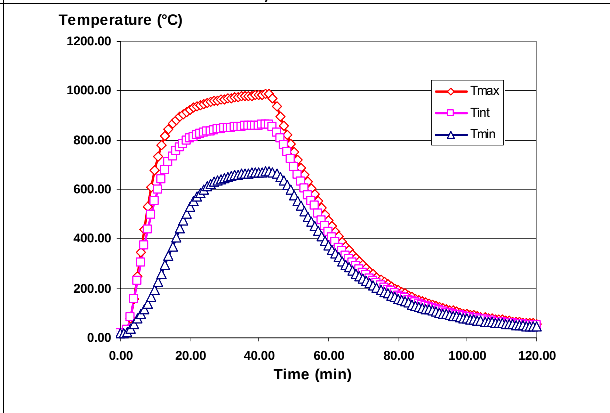
a) Z=3.5m



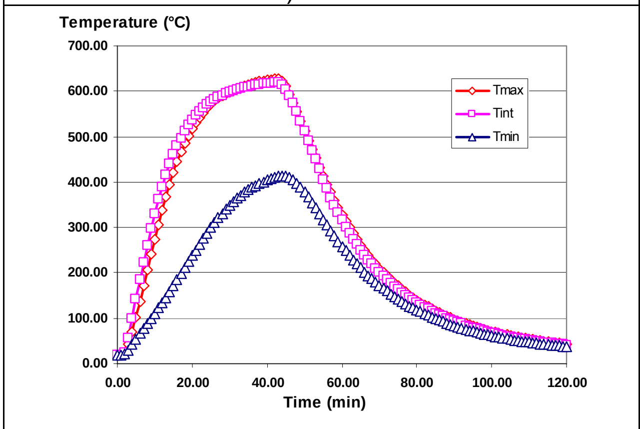
b) Z=4.5m



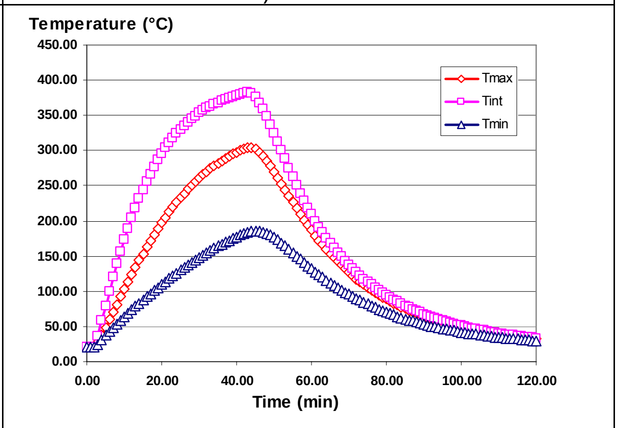
c) Z=5m



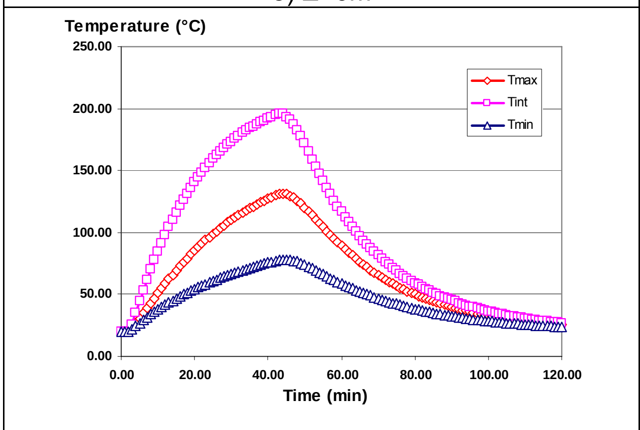
d) Z=5.5m



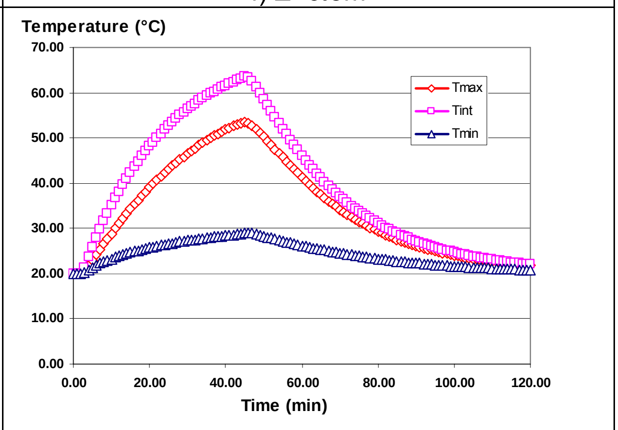
e) Z=6m



f) Z=6.5m



g) h=7m



h) h=7.5m

Figure 7: Example of temperature development in SHS column 150x8mm

Table 1: Temperatures field at failure of some external columns with stainless steel engulfed in fire

Cross-section		Compartment Size: 3×4m (fire load density 1500MJ and 50% of windows opening)			Compartment Size: 9×8m (Fire load density 1500MJ and 50% of windows opening)		
		$\eta_{fi,t}$=0.3	$\eta_{fi,t}$=0.5	$\eta_{fi,t}$=0.7	$\eta_{fi,t}$=0.3	$\eta_{fi,t}$=0.5	$\eta_{fi,t}$=0.7
RHS 150x8	T_{max} (exposed side)	N.F*	980	870	N.F*	921	813
	T_{int}(intermediate side)	N.F*	738	566	N.F*	783	690
	T_{min} (unexposed side)	N.F*	534	234	N.F*	361	214
	$T_{mean} = (T_{max}+2T_{int}+T_{min})/4$	-	748	559	-	712	602
	Failure time (min)	-	**38.2**	**15.3**	-	**20.4**	**14.7**
RHS 300x8	T_{max} (exposed side)	N.F*	N.F*	883	N.F*	945	833
	T_{int} (intermediate side)	N.F*	N.F*	576	N.F*	802	696
	T_{min} (unexposed side)	N.F*	N.F*	242	N.F*	392	208
	$T_{mean} = (T_{max}+2T_{int}+T_{min})/4$	-	-	571	-	735	596
	Failure time (min)	-	**-**	**15.8**	-	**22.4**	**15**

* Column remains stable during all fire exposure
$\eta_{fi,t}$ is the load level under fire situation

Table 2: Temperatures field at failure of some external columns with carbon steel engulfed in fire

Cross-section		Compartment Size: 3×4m (fire load density 1500MJ and 50% of windows opening)			Compartment Size: 9×8m (Fire load density 1500MJ and 50% of windows opening)		
		$\eta_{fi,t}$=0.3	$\eta_{fi,t}$=0.5	$\eta_{fi,t}$=0.7	$\eta_{fi,t}$=0.3	$\eta_{fi,t}$=0.5	$\eta_{fi,t}$=0.7
RHS 150x8	T_{max} (exposed side)	908	602	467	877	686	538
	T_{int} (intermediate side)	768	601	514	709	586	503
	T_{min} (unexposed side)	451	115	75	320	177	125
	$T_{mean} = (T_{max}+2T_{int}+T_{min})/4$	699	480	392	654	509	417
	Failure time (min)	25.1	9.9	7.8	**11.2**	**8.7**	**7.6**
RHS 300x8	T_{max} (exposed side)	794	593	423	862	600	449
	T_{int} (intermediate side)	750	583	481	700	539	448
	T_{min} (unexposed side)	390	106	63	293	132	91
	$T_{mean} = (T_{max}+2T_{int}+T_{min})/4$	670	471	362	623	434	339
	Failure time (min)	20.1	8.7	6.6	**10.8**	**8.0**	**7.0**

* Column remains stable during all fire exposure
$\eta_{fi,t}$ is the load level under fire situation

4.2 DEVELOPMENT OF DESIGN GUIDANCE

4.2.1 Main features of simple calculation rules

The buckling resistance of stainless steel external columns under axial compression (class of cross section ≤ 3) in the fire situation can be obtained from:

$$N_{b,fi,t,Rd} = \chi_{fi}(\bar{\lambda}_\theta).\sum_{i=1}^{4} A_i.f_{2,\theta i} / \gamma_{Mfi} \tag{1}$$

where:

- A_i is the area of plane element i defining the hollow cross-section (exposed side, lateral side or unexposed side);
- θ_i is the temperature of plane element i calculated from the simplified 2D heat transfer model developed for hollow steel section;
- $f_{2,\theta i}$ is the 2% proof characteristic strength at temperature θ_i;
- $\gamma_{M,fi}$ is the partial factor for the fire situation;
- $\bar{\lambda}_\theta$ is the non dimensional slenderness at elevated temperature θ;
- χ_{fi} is the reduction factor for flexural buckling in the fire design situation obtained from an appropriate buckling curve and depending on the non-dimensional slenderness:

The reduction factor χ_{fi} for buckling resistance in the fire design situation is determined according to:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^{\ 2} - \bar{\lambda}_\theta^{\ 2}}} \qquad \text{with} \qquad \chi_{fi} \leq 1.0 \tag{1}$$

With help of the buckling curve defined in EN 1993-1-2, there is $\varphi_\theta = 0{,}5(1 + 0{,}65\bar{\lambda}_\theta \sqrt{\frac{235}{f_y}} + \bar{\lambda}_\theta^{2})$.

The non dimensional slenderness $\bar{\lambda}_\theta$ at temperature θ is given by:

$$\bar{\lambda}_\theta = \sqrt{\sum_i A_i \ f_{y,\theta i} / N_{fi,cr}} \tag{2}$$

where $N_{fi,cr}$ is the Euler elastic critical load effective flexural stiffness obtained from:

$$N_{fi,cr} = \pi^2 (EI)_{eff} / l_\theta^{\ 2} \tag{3}$$

Where l_θ is an equivalent buckling length in fire situation taking into account effects of thermal gradient and column continuity on the fire resistance of the column. For an intermediate storey the buckling length l_θ of a continuous column may be taken as $l_\theta = L - h_s$ (with h_s the height of sill) and at the top storey the buckling length may be taken as $l_\theta = L$, where *L* is the system length in the relevant storey.

The effective rigidity, $(EI)_{fi,eff}$, is determined as follows:

$$(EI)_{fi,eff} = \sum_i E_{i,\theta} . I_i \tag{4}$$

where:

- $E_{i,\theta}$ is the modulus of elasticity of plate element i at the appropriate elevated temperature θ_i;
- I_i is the second moment of area of plate element i;

Obviously, checking of the external column consists in satisfying the condition:

$$N_{fi,Ed} \leq N_{fi,Rd} \tag{5}$$

where $N_{fi,Ed}$ is the design value of the axial compression for the combination of actions considered in fire situation (according to EN 1991-1-2).

4.2.2 Comparison of results between simple design rules and numerical model

To show the accuracy of the simple calculation method described in the previous paragraph, for all members investigated in our parametric study, critical temperatures calculated with simple calculation rules and those obtained numerically with the computer code ANSYS have been compared. It has to be noted here that the critical temperature of stainless steel columns is defined as the mean temperature obtained from the more severe temperature distribution over the column cross-section at failure. Moreover, comparison between the above two approaches is based on the load level derived from design resistance so the same absolute load values. The reason of doing so is that the applied load under fire situation will be based on simplified calculation rules only.

The results of such comparison are illustrated in **Figure 8**, with the numerical results on X-axis and the simplified calculation results on Y-axis. It can be seen that simplified calculation rules agree quite well with the numerical model. The difference between the simplified calculation rules and the numerical model does not exceed 10 %, with some points on the unsafe side.

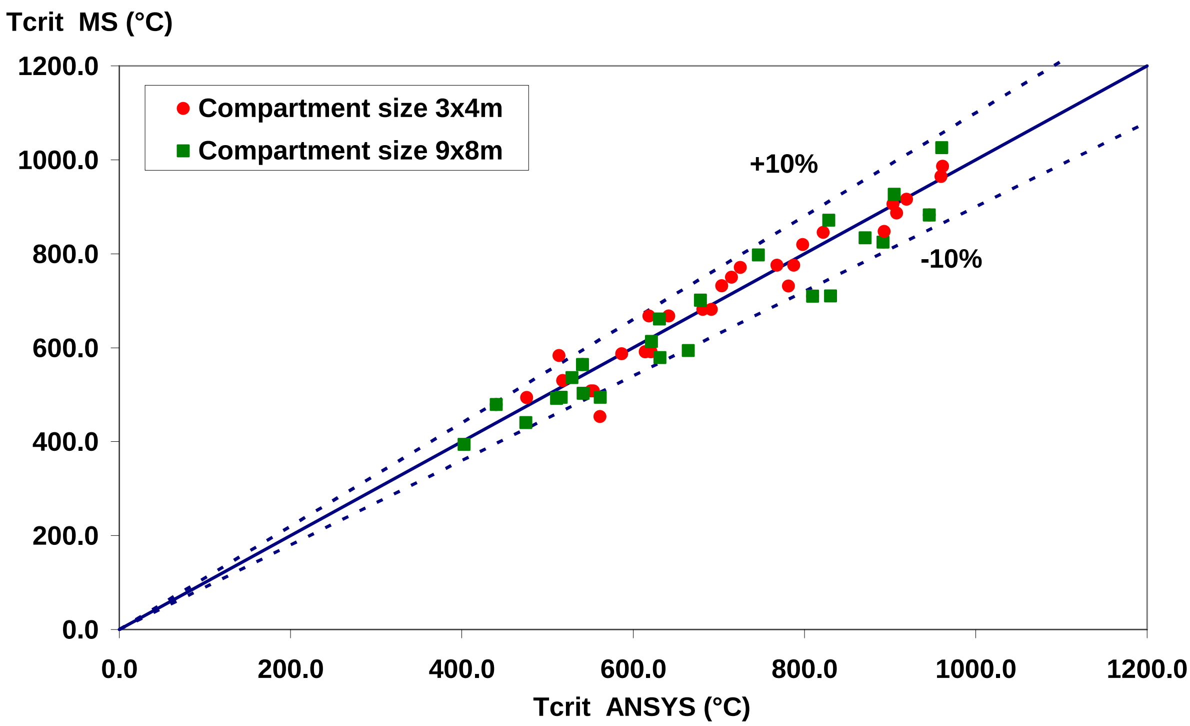


Figure 8: Comparison of critical temperatures calculated using simplified method (T_{crit} MS) and numerical model (T_{crit} ANSYS)

In conclusion, the proposed simple calculation rules (combining a simplified 2D heat transfer model to calculate the temperatures versus time of the four sides of the hollow section with the previous mechanical model) are fully suitable to predict with a good precision the fire resistance of external stainless steel members under axial compression.

5. CAR PARK BUILDINGS

Another aim of this project research was to investigate the potential application of unprotected stainless steel members under localised natural fire conditions. In this purpose, it has been proposed to study the behaviour of steel and concrete composite structures of open car parks with stainless steel columns using a fire safety engineering procedure developed in France and validated against experimental results. In fact, this engineering procedure was already applied to open car parks with unprotected carbon steel and concrete composite structures. It led to the establishment of design guidance giving the minimum size of unprotected carbon steel members as well as the maximum load ratio of carbon steel column to be used for different framing systems commonly used in open car parks in France. Adopting the same framing systems, numerical investigations have permitted to define the maximum load level for unprotected columns with stainless steel hollow section.

In following section, the principle of above mentioned fire engineering procedure applied to open car parks and main corresponding results are given.

5.1 FIRE ENGINEERING PROCEDURE APPLIED TO OPEN CARS PARKS

The fire behaviour of open car parks has been investigated according to a fire engineering procedure based on natural fire concept and composed of following steps:

- Determination of the most unfavourable fire scenarios with respect to the fire stability of the structure and according to the arrangement of car park position (number of cars involved in fire and their positions). In theory, the fire behaviour of open car parks should be checked under the three basic fire scenarios indicated on Figure 9. These scenarios consider that seven vehicles on the same parking line, four vehicles in two successive parking lines or only one vehicle at any position of car park are involved in fire. However, according to the objective of actual study, only one fire scenario in relation to four vehicles is considered in the present study. This fire scenario assumes that a column is engulfed fully in a fire of four vehicles around. The fire starts in one of four vehicles and spreads to three vehicles around. The vehicles are in class 3 (which corresponds to model of type Laguna, Mondeo, Passat, Vectra, etc) except one of the vehicles which is of type utilitarian. In this case, its heat release is much more important.

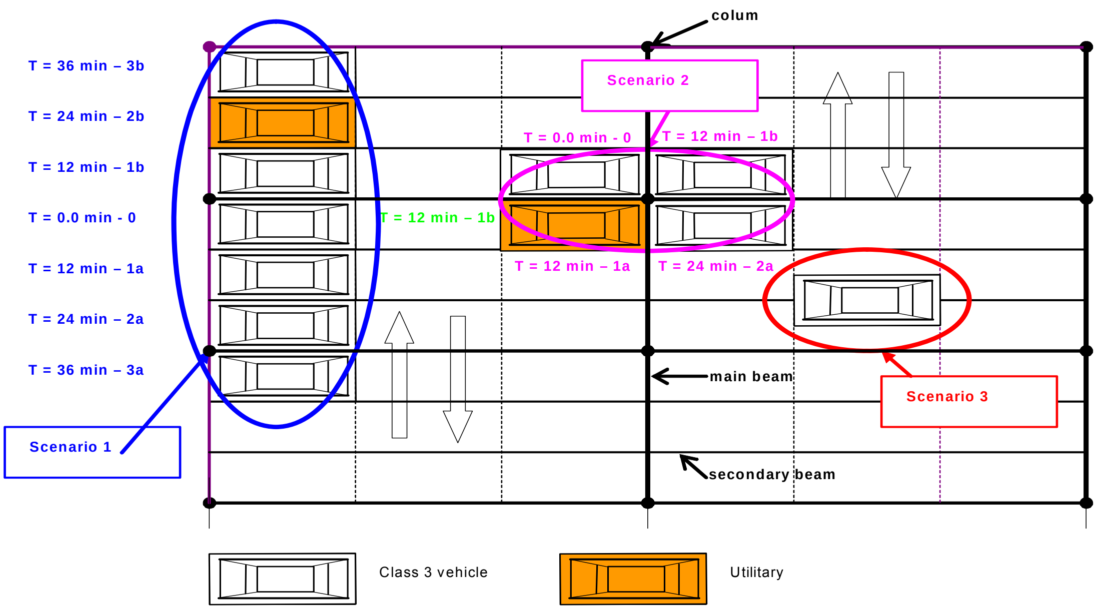


Figure 9: Basic fire scenario for open car parks

- Evaluation of thermal actions as a function of time applied to different structural members according to the fire scenario and corresponding fire development based on the heat release rate of different vehicles as well as the propagation of fire between them. For open car parks, structural members near fire are generally subjected to heat flux derived from fire flames while structural members far from fire will be only heated slightly by the layer of hot gas. Therefore, in some case the thermal action for structural members will be the combination of both of them, As example, **Figure 10** shows the heat release rate of four vehicles involved in fire

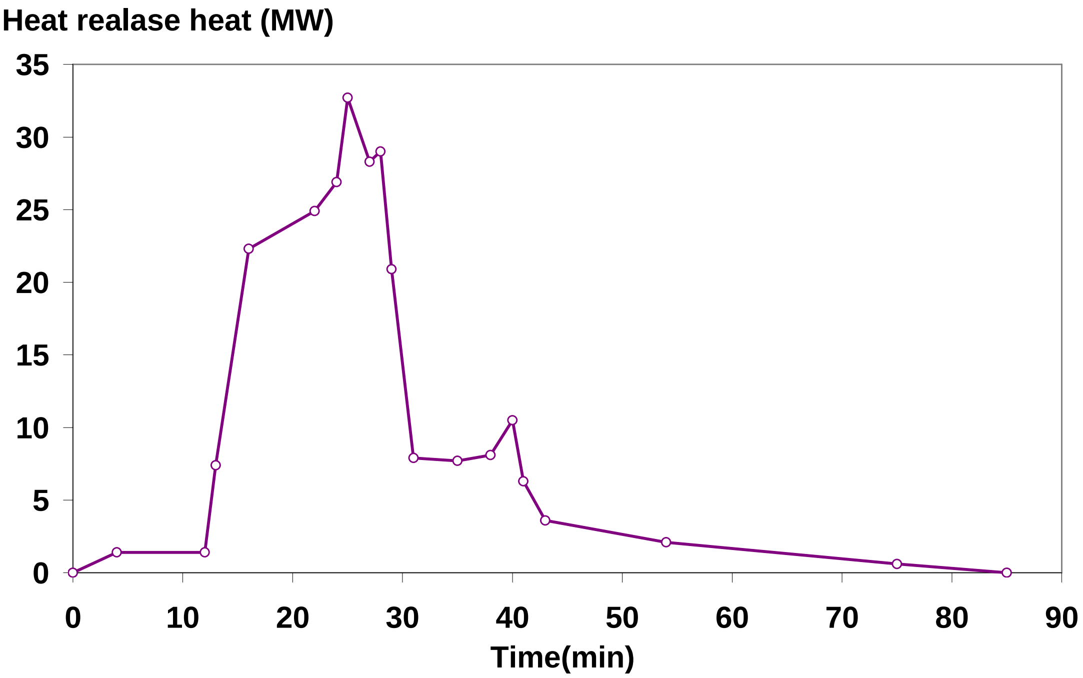
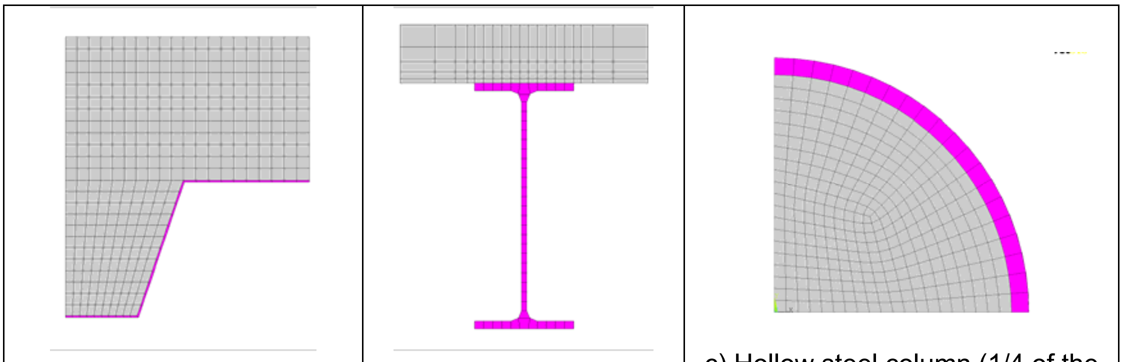


Figure 10: Heat release rate of 4 vehicles involved in fire

- Estimation of heating of structural members considering possible temperature gradient not only on the cross section but also along the member's length. It has been noted that this analysis needs to use advanced calculation model. In the present study, the computer code ANSYS has been employed. The four main structural members and corresponding finite element mesh are illustrated in Figure 11**.** Only one half of the composite slab and one quarter of the column was modelled to reduce the size of the model. Each structural member was considered to be thermally independent and its connection with other members was neglected.



a) Composite slab

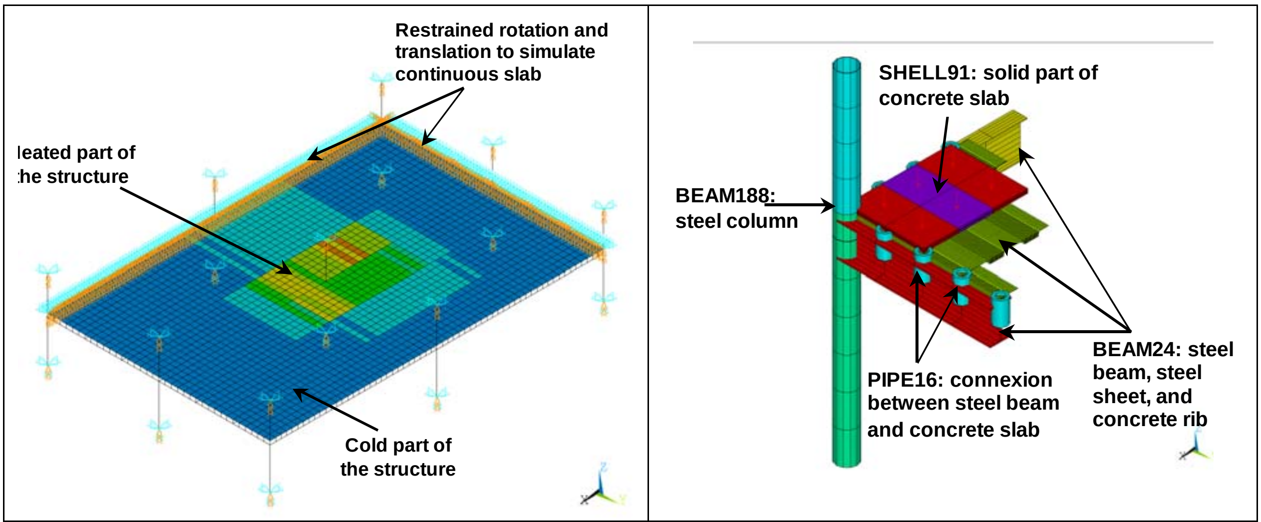
b) beams

c) Hollow steel column (1/4 of the cross-section)

Figure 11: Meshing of main structural members for thermal analysis

- Calculation of mechanical behaviour of car park structure based on a 3D numerical modelling with previously predicted structural heating. This global analysis allows taking into account lateral buckling of steel beams, membrane and diaphragm effects on the floor and load redistributions from the heated part of the structure to cold parts. In our parametric study, the computer package ANSYS has been also employed using different types of finite elements in modelling. As shown in **Figure 12**, the fact to have a locally heated part of the structure gives the possibility of modelling only a small part of the car park. The reason of this modelling strategy is to reduce to minimum the computation cost which becomes very important if a large area of floor is taken into account. In this case, the influence of remained part of the structure is represented by appropriate boundary conditions. Details of numerical modelling are also given in **Figure 12**. In this modelling, following four types of finite elements have been used: 3D non-linear line element - BEAM24, 3D non-linear line element beam 188, 3D non-linear multi-layer shell element – SHELL91 and 3D linear line element – PIPE16. In particular, it can be underlined that the composite floor is represented by following finite elements: shell element for solid part of composite slab as well as reinforcing steel grid; beam-column element for steel members, steel sheet and ribs of composite slab and rigid link element for full connection between steel beams and composite slab. Moreover, because it is only a part of the structure, there are some restrained conditions from non-modelled part of the structure to be taken into account. These restrained conditions are represented by equivalent boundary conditions: fully fixed column bases due to continuous column condition and lower floor staying cold and rotation and lateral displacement restrained slab because of the continuity condition of the slab. Another important aspect to be mentioned here is the material mechanical properties at elevated temperatures. In our study, recommendations given in EN 1994-1-2 for this feature are followed closely for steel and concrete. In addition, the concrete is considered as strength irreversible material which means that as far as it is highly heated it will not recover its initial strength during the cooling phase. This material property is certainly closer to reality and has to be taken into account in order to obtain realistic structure behaviour in natural fire condition. Although the numerical modelling given above is already quite complicated, it is still necessary to check some specific features which are not included in numerical model such as the possible rupture of reinforcing steel due to too important elongation or the movement of vehicles, if deflection of floor becomes too high, modifying initial loading condition of floor. As a consequence, two special criteria have been systematically checked for numerical results of all structure frames, which correspond to:

- maximum mechanical strain of reinforcing steel not exceeding 5%
- maximum deflection of floor not higher than 20th of secondary beam span



a) Modelled structure	Detail of numerical modelling

Figure 12: Example of modelled structure and boundary conditions

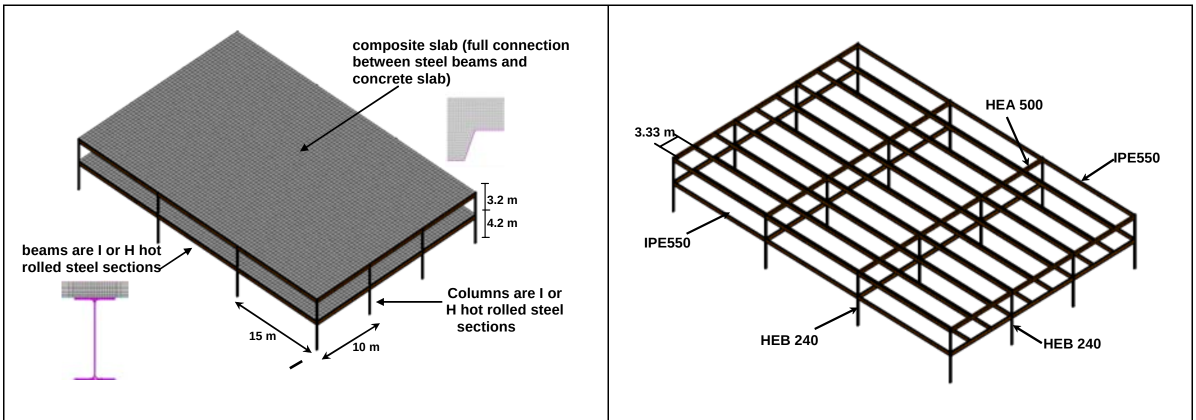
5.2 NUMERICAL ANALYSIS OF OPEN CAR PARKS

5.2.1 Investigated open car parks

Two different structures corresponding to standard framing systems commonly used in open car parks in France were investigated in our study. They correspond to two-level steel and concrete composite structures composed of composite floor system (steel beams connected with composite slab) and steel column partially protected by concrete (see **Figure 13**). The structural detail of these standard frame systems using carbon steel are reported in table 1. For this structure, the length direction of parking places is parallel to secondary beams.

Frame structure		Case 1	Case2
Level height (m)	First level	4.10	3.10
	Second level	2.67	2.67
Cross section of members (standard level)	Column	HEB 240 (S355)	HEA 340 (S355)
	Main beam	HEA 500 (S355)	IPE 400 (S355)
	Secondary beam	IPE 500 (S355)	IPE 240 (S355)
	Composite slab	Cofraplus60	Cofraplus60
Span of secondary beams (m)		15	7.5
spacing of secondary beams (m)		3.33	2.50
Width of parking place (m)		2.5	2.5
Number of vehicles between two successive secondary beams		1	1
Spacing of columns (m)		10.0	7.50
Span of main beams (m)		10.0	7.50
Total depth of concrete slab (cm)		12	12

Table 3: Summary of studied structure frame systems



a) first standard structure frame | b) Corresponding steel frames

Figure 13: Detail of the first standard frame systems of open car parks

5.2.2 Preliminary design of stainless steel columns

Before starting the analysis of the fire behaviour of the two selected car park structures, preliminary calculations have been carried out to design the minimum CHS columns with stainless steel (leading to the maximum load ratio available with stainless steel) to be used in place of hot rolled H carbon steel columns given in Table 3. This design has been made as follows:

1. Evaluation of a stainless steel column having the same buckling resistance at room temperature design than the partially encased carbon steel section used in open car park. In

case of investigated open car parks, the load level of carbon steel columns is 0.35.

2. Determination of the temperature development in the stainless steel column (considering temperature gradient along the member's length) and estimation of the column buckling resistance as function of time for the critical cross-sectional temperature previously obtained. In fact, these results give the maximum load level (defined as the ratio of the fire buckling resistance over the ultimate load at room temperature design) available for columns.
3. Design of the stainless steel column cross-section so that the column load level (defined as the ratio of the applied load under fire situation over the ultimate load at room temperature design) becomes equal to the previously obtained value.

Stainless steel columns showing the same room temperature buckling resistance than carbon steel columns are reported on Table 4. It is noticed that room temperature buckling resistances of carbon steel columns have been calculated according to EN 1993-1-1. Buckling resistance of stainless steel columns have been evaluated according to EN 1993-1-4 using the buckling parameters given for cold-formed sections (i.e. α=0.49 and $\overline{\lambda}_0$=0.40).

Framing systems	Carbon steel column		Stainless steel column	
	Cross-section	Room temperature Buckling Resistance (KN)	Cross-section	Room temperature Buckling Resistance (KN)
N°1	HEB240 (S355) L=4.1m	2741	CHS 323.9x12mm	2800
N°2	HEB340 (S355) L=2.67m	5479	CHS 610x12mm	5410

Table 4: Preliminary design of stainless column

As example, Figure 14 shows temperature distributions calculated along the height of the CSH column b=356x12mm. To show the benefit of filling concrete on the thermal behaviour of column, comparison has been made with geometrical identical steel column. As expected, it can be noted that simple steel column is heated faster than the composite column. After 30 minute exposure to real fire, the maximum temperature was 720°C for composite column in comparison to 900°C for steel column, which gives a temperature difference of about 200°C. After 60 minutes, this difference reduces to 30°C with cross-sectional temperatures of 430 C and 460°C for composite column and steel column respectively.

From the maximum heating obtained previously, if analysis had been made considering an isolated column, the conclusion that the collapse is inevitable could have been easily drawn since according to EN 1993-1-2, at 800 °C, the residual strength of stainless steel represents only 27% of its initial strength at room temperature while in general the load level of steel members under fire situation is often around 50%. However, for open car parks, fire affects directly only a limited area leading to an important loss of strength of corresponding structural members at this location while when moving away from fire source, the relevant parts of structural members remain still resistant enough since their heating is limited to only around 200°C. Considering the fact that under fire situation the structure should resist a design load equal to about half of its ultimate load at room temperature, the cold part of structure is capable of taking over more load under fire situation. So, taking into account the load redistribution between both heated and cold parts of the structure, unprotected steel structures can remain stable in case of car fire.

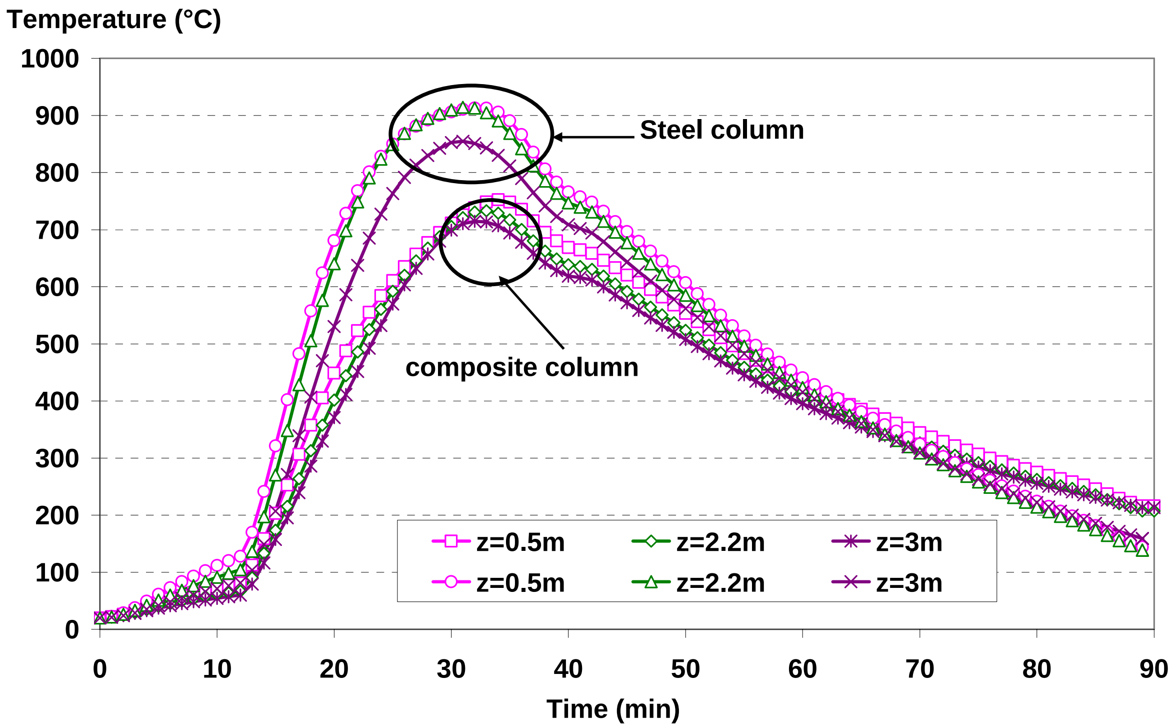


Figure 14: Heating of CHS column b=323.9x10mm

Figure 15 shows the evolution of the load level of both composite and steel columns as a function of time. Load levels were defined here by the ratio between the fire buckling resistance of column calculated according to the previously critical cross-sectional temperature with the CTICM method for cold worked stainless steel (i.e. buckling parameters □=0.49 and $\bar{\lambda}_0$ =0.40 and using the 0.2% proof characteristic strength of steel) divided by the design room temperature buckling resistance. It is assumed here that the filling concrete does not contribute to the load bearing capacity of columns. Obviously, according temperature rises given in **Figure 14**, load levels decreases rapidly during the heating stage, reach a minimum value after 32 minutes of fire exposure and then increase during the cooling phase. It can be noted than the minimum load level (corresponding in fact to the to available maximum load level for columns) of the composite column is higher than this obtained for steel column. This value is close to 0.45 in comparison to 0.2 for identical steel column.

Theses results clearly indicate that bare stainless steel columns can not be used economically in open car parks. Indeed, their maximum load level is lower than the maximum value given for partially concrete encased carbon steel column, i.e 0.35. Stainless steel columns should be filled with concrete to limit their temperature rise.

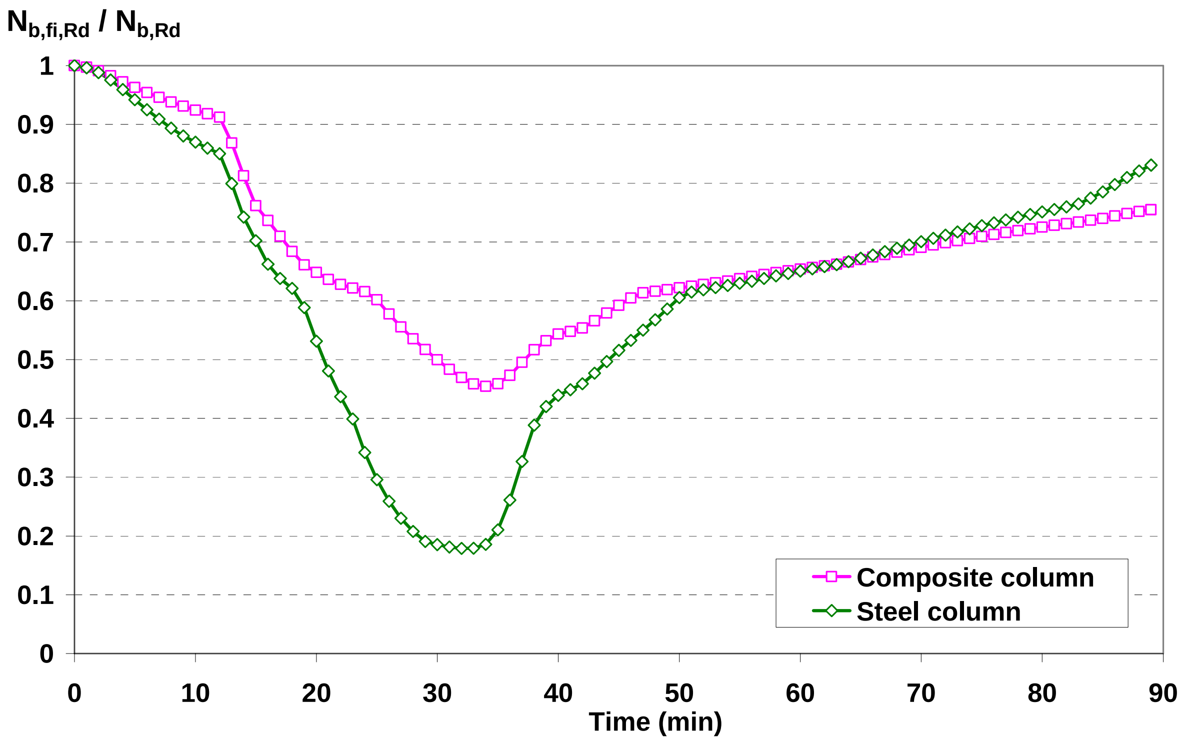


Figure 15: Heating of CHS column b=323.9x12mm

Finally, stainless steel columns designed for a load level of 0.45 are reported on **Table 5**.

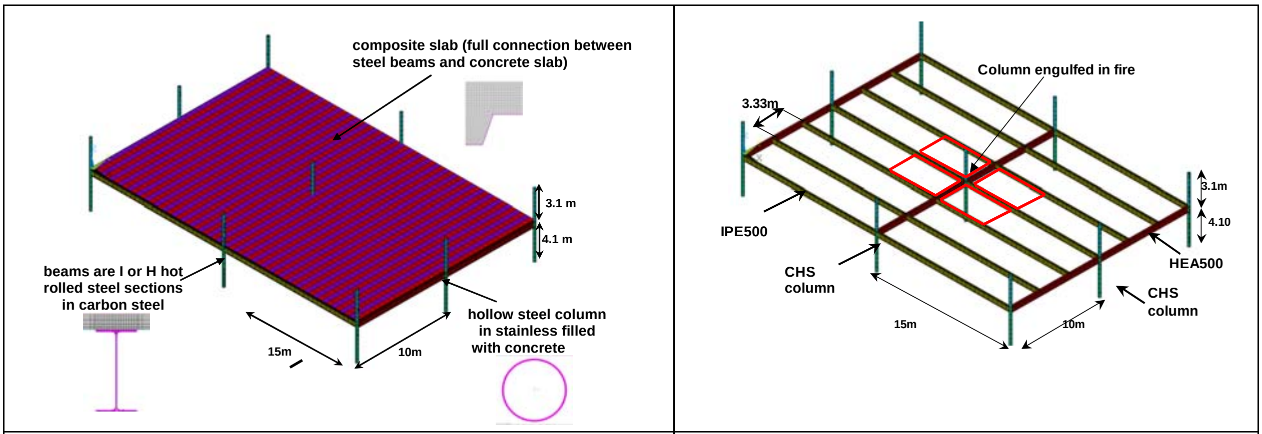
Framing systems	Stainless steel column	
	Cross-section	Room temperature Buckling Resistance (KN)
N°1	CHS 273x12mm	2235
N°2	CHS 407x14mm	4148

Table 5: Buckling resistance at room temperature

5.2.3 Mechanical behaviour of car park structures

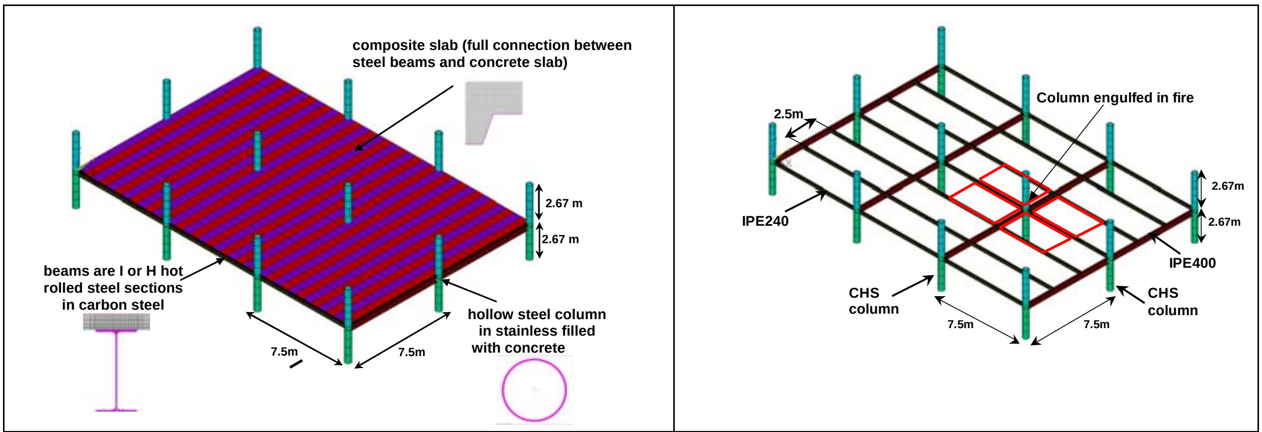
Structural behaviour of two selected open car parks (see Table 3) with previously predicted stainless steel columns has been investigated according to the engineering procedure described above.

In our study, the four vehicles involved in fire were considered to situate around a column of the lower level of the structure. Modelled structures as well as fire location for this fire scenario are given in Figure 16 and Figure 17. In fact, only four frames and six frames have been taken into account for the first and the second open car parks respectively. Floors of the upper level have been neglected in modelling and replaced by appropriate boundary conditions. Moreover, because only a part of the structure is modelled, restrained conditions have been introduced to simulate continuous concrete slab. (see
Figure 18).



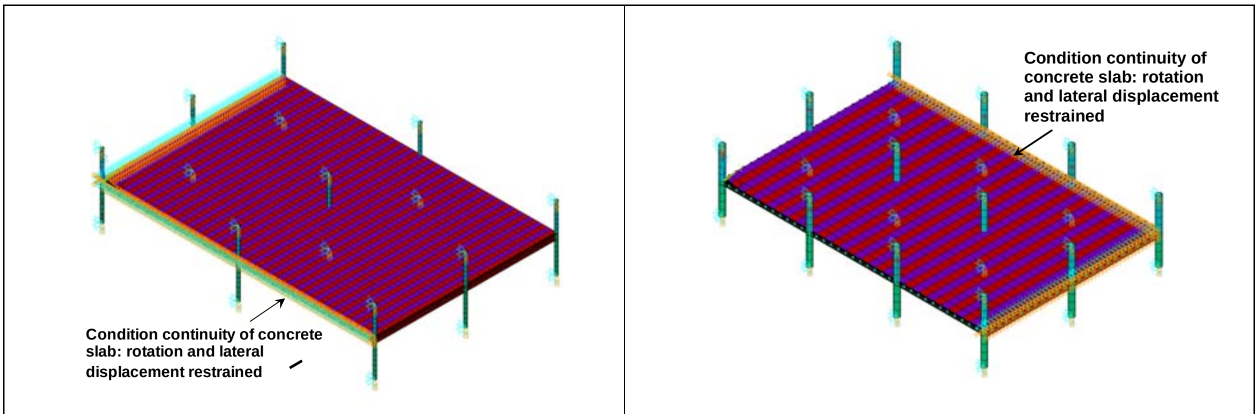
a) Modelled structure | b) Corresponding steel frames

Figure 16: Detail of first standard frame systems



a)Modelled structure | b) Corresponding steel frames

Figure 17: Detail of second standard frame systems



a) first standard frame systems | **b) second standard frame systems**

Figure 18: boundary condition to modelled cold part of the structure

The other parameters assumed in present study are the following:

- Concrete compressive resistance: 30 MPa;
- Steel grade for structural carbon steel members: 355 MPa;

- Steel grade EN 1.4404 for stainless steel. Material properties at room temperature were taken according to EN1993-1-4 with following nominal values at room temperature: f_y = 240 N/mm², f_u = 530 N/mm², E = 200,000 N/mm²
- Steel grade for reinforcing steel: 500 MPa;
- A reinforcing steel mesh of 150 mm x 150 mm with a diameter of 7 mm;
- imposed load of 140 kg/m² for floor under fire situation in addition to dead load from structure, light system (20 kg/m²), waterproof layer (light layer for common level – 5 kg/m² and heavy layer for top layer – 145 kg/m²) and heavy façade (750 kg/m);
- full shear connection between steel beams and concrete slab with help of welded headed studs.
- Stainless steel column are hollow steel section filled with concrete. The concrete here is considered to play a role of thermal insulation without any contribution to mechanical resistance of columns.

The loads (in fire combination) of the first level are uniformly distributed on the concrete slab and the resultants of the loads applied on the upper concrete slab are applied on the top of each column (see Figure 19).

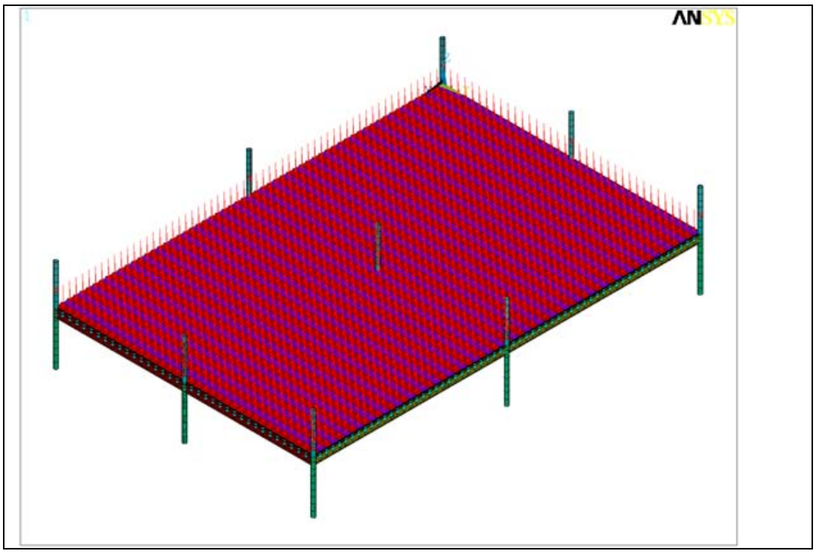


Figure 19 : Distributed loads of the structure

Figure 20 shows the heated parts of the two investigated open car parks. As example maximum heating of four different structural members and temperatures rise calculated along the most heated beams are given on Figure 21, Figure 22 and Figure 23 respectively for the first open car parks. It can be seen that the temperature of beams reaches locally more than 900°C.

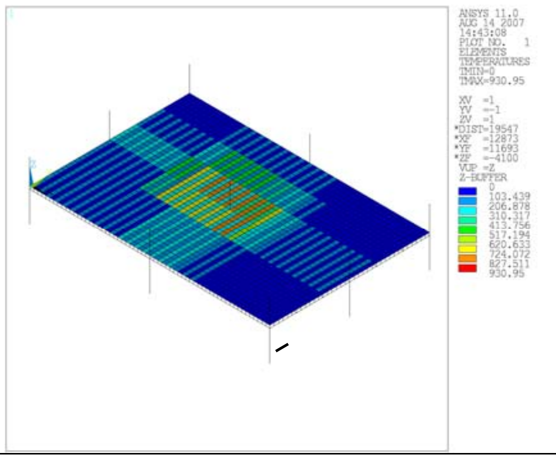
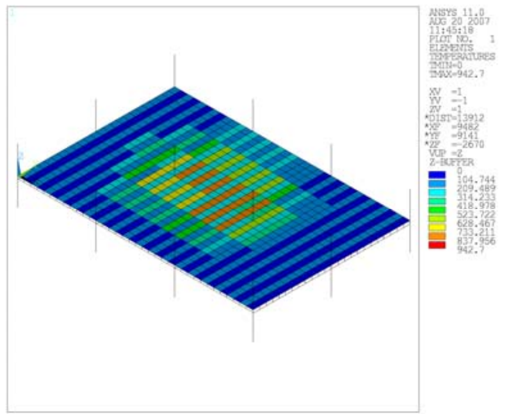
	
a) first standard frame systems	b) second standard frame systems

Figure 20: Heated parts of the modelled structures

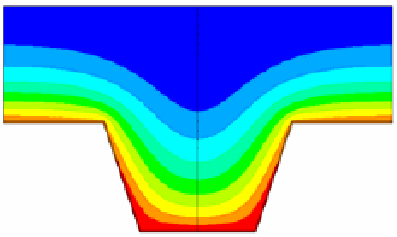
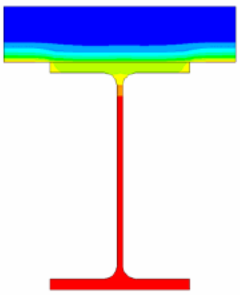
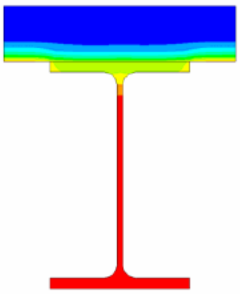
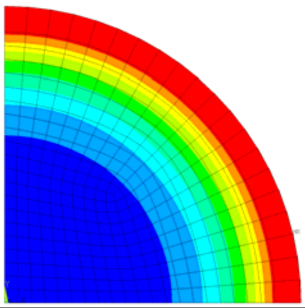
	
a) composite slab	b) Main beam in HEA 500
	
e) secondary beam in IPE 550	d) CHS column filled with concrete

Figure 21: Maximum heating of different structural members in the first open car parks

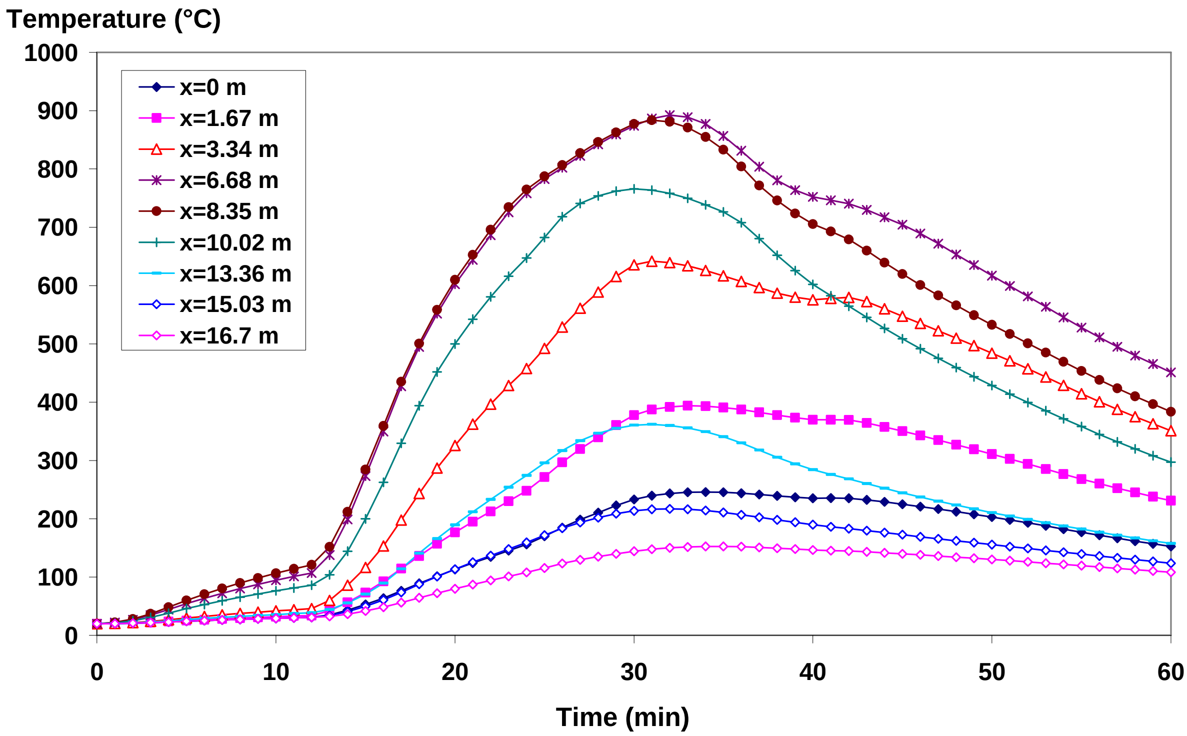


Figure 22: Temperature rises along the most heated main beam

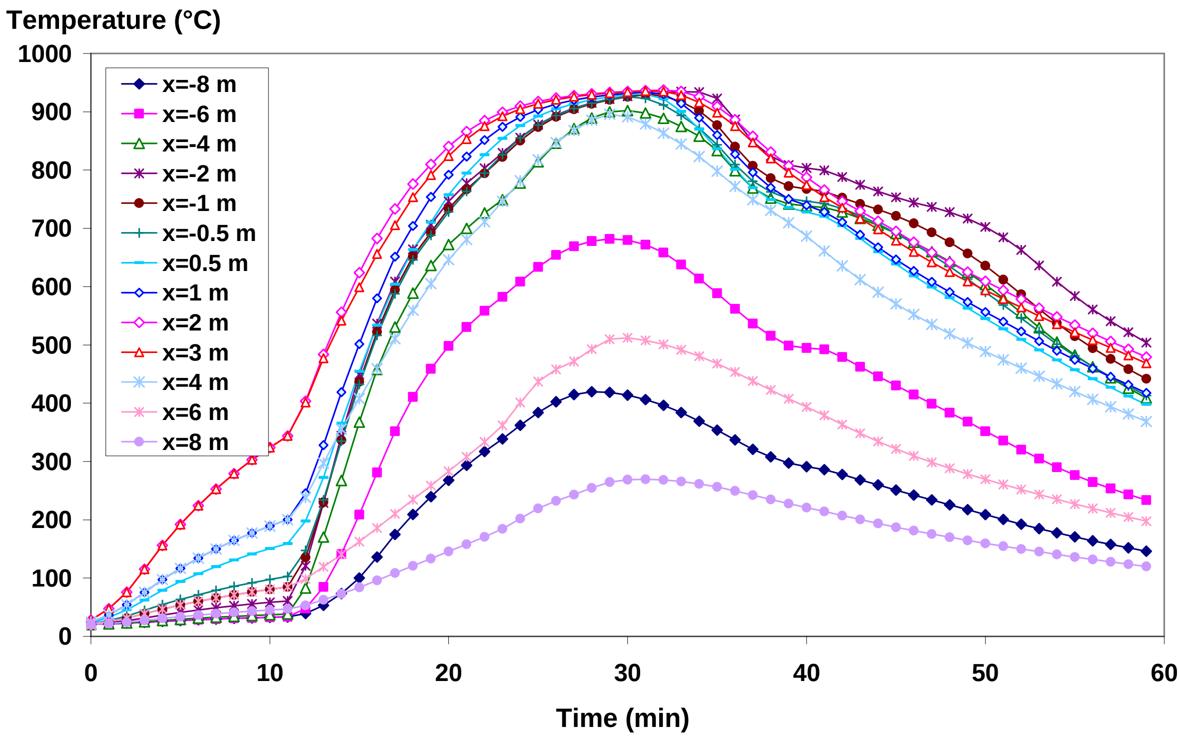


Figure 23: Temperature rises along the most heated secondary beam

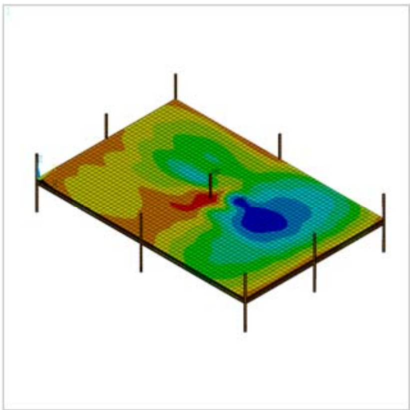
Figure 24 gives the deformed state of the first investigated structure at different fire instants as well as maximum displacements of the most heated beams.

This figure illustrates very well the consequence of the localised fire development on the displacement behaviour of the floor of which as example maximum vertical deflection increases from 52 mm at 15 minutes of fire exposure to 430 mm after 33 minutes of fire. Then at 60 minutes of fire, the maximum deflection of the floor decreases to 395 mm but the deformed area increases because of more involved area by the fire development. The decrease of deflection is due to the fact that the fire has passed its maximum heating phase and entered the cooling phase. Concerning the maximum deflection of steel beams, it can be found that it is close to 412 mm for secondary beams and 318 mm for primary beams which is far away from the defined failure criteria limiting the maximum deflection to 20th of the span (namely 750mm for secondary beam and 500 for main beam). From this point of view, the performance of the first open car park can be considered as fully satisfying under corresponding fire scenario.

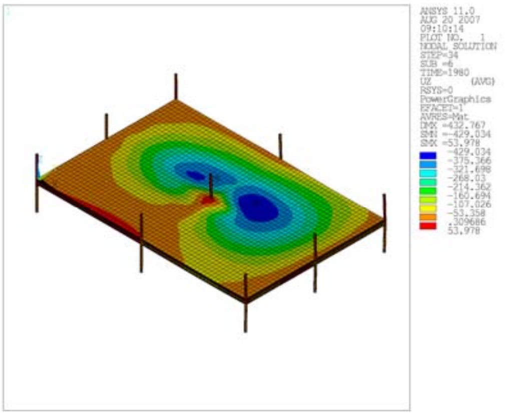
Another failure criterion to be investigated for above modelled structure is the elongation of reinforcing steel grid in the composite slab (see **Figure 24**). It has been considered that the maximum elongation of reinforcing steel shall not exceed 5%, which, in fact, corresponds to the minimum value of elongation capacity of all types of reinforcing steel specified in EN1992-1-2 (fire part of concrete structure). Moreover, these failure criteria have been validated in two of ECSC projects through the numerical modelling of fire tests in real buildings. For actual example, the maximum elongation of reinforcing steel grid obtained in numerical simulation is 4.2% so less than 5%. Therefore, this failure criterion is also fully satisfied.

Figure 25 gives the deformed states of the second investigated structure as well as the maximum displacements of the most heated beams. It can be easily seen that the maximum deflection of steel beams (namely 127mm for secondary beams and 44mm for mean beams) are lower than the defined failure criteria of span/20. The maximum elongation of reinforcing steel grid doesn't exceed also the failure criterion of 5%.

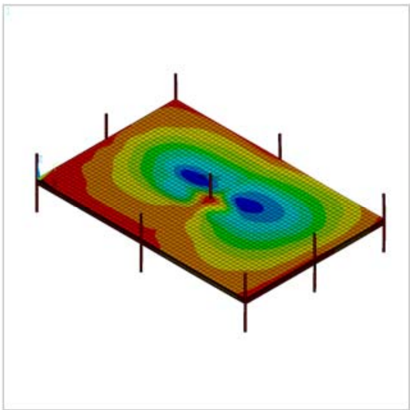
In conclusion, the results of numerical simulations confirm that the fire stability of the two open car parks is satisfactory for selected fire scenarios. Moreover, stainless steel columns filled with concrete are able to support the applied loads during all the time of fire.



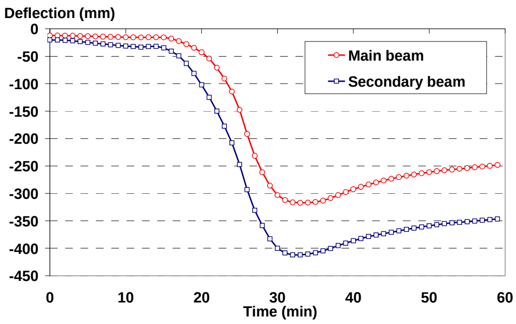
a) Deformed state of floor at 15 min of fire



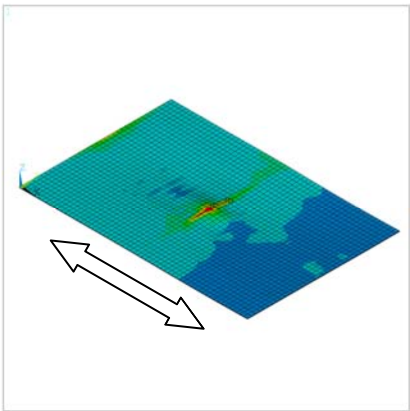
b) Deformed state of floor at 33 min of fire



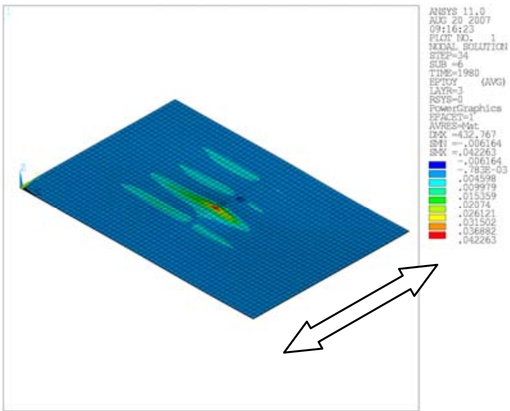
c) Deformed state of floor at 60 min of fire



d) Deflection of steel beams versus time

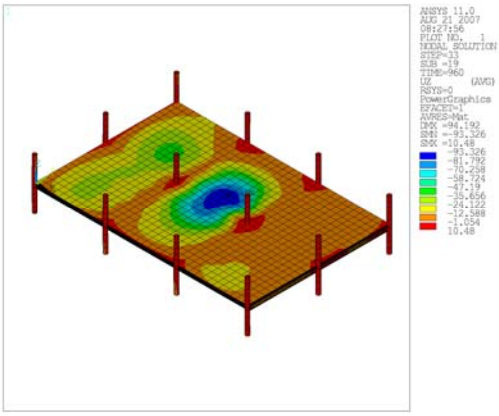


e) Strain of reinforcing steel // slab span

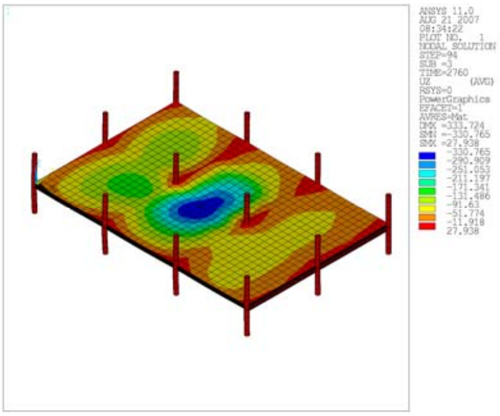


f) Strain of reinforcing steel ⊥ slab span

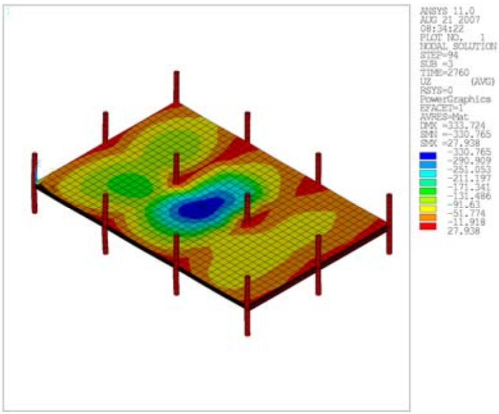
Figure 24: Calculation results for first open car park



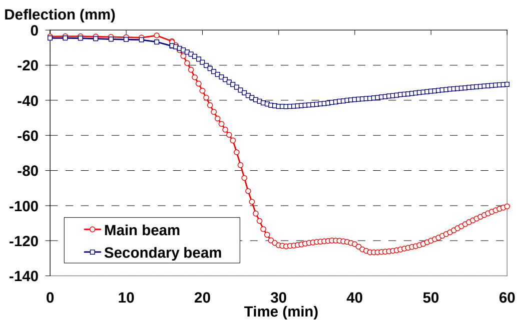
a) Deformed state of floor at 16 min of fire



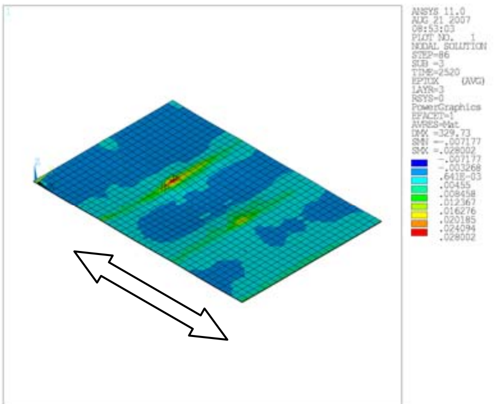
b) Deformed state of floor at 46 min of fire



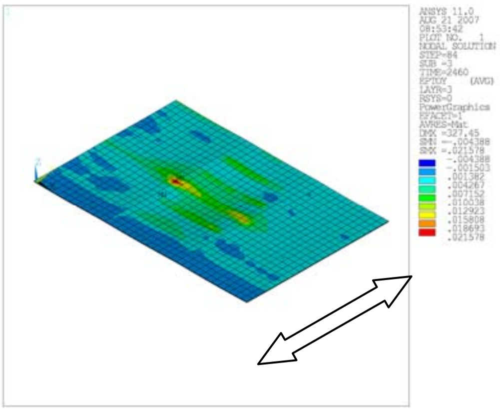
c) Deformed state of floor at 60 min of fire



d) Deflection of steel beams versus time



e) Strain of reinforcing steel // slab span



f) Strain of reinforcing steel ⊥ slab span

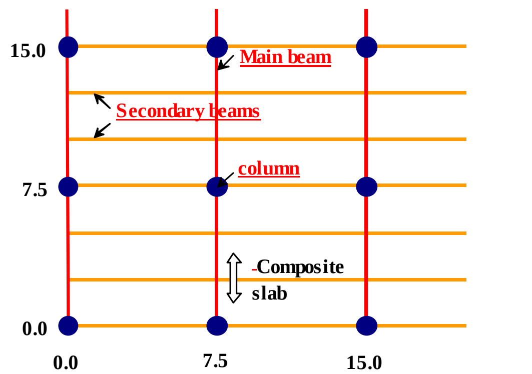
Figure 25: Calculation results for second open car park

5.2.4 Conclusion

Numerical analysis performed on standard open car parks subject to natural fires has shown that the maximum load level of unprotected stainless steel columns with steel grade E.N14404 is 0.45. This value is higher than the maximum load level already established for partially concrete encased carbon steel columns, i.e. 0.35 which is more expensive to build. So the use of stainless steel could allow the employment of reduced column cross-sections in comparison to carbon steel column. Stainless steel columns should be filled with concrete to limit their temperature rise and to ensure their stability during fire.

5.5 DEVELOPPEMENT OF DESIGN GUIDANCE

Numerical results have allowed to extend the design tables already established for different standard framing systems of open car parks using unprotected carbon steel and composite structures to the employment of stainless steel hollow column filled with concrete. One example of such design table is given below (see table 6). Other design tables as well as construction details to be used are regrouped together in a practical design guide [9].



Slab span: 2,5 m
Secondary beam span : 7,5 m
Main beam span: 7,5 m
Spacing of columns : 7,5 m

Applied loads (except self weight):
Standard level:
- Dead load: 0,20 kN/m²
- Imposed load: 2,50 kN/m²

Last level:
- Dead load: 1,45 kN/m²
- Imposed load: 2,50 kN/m²

Self-weight of facade: 7,5 kN/m²

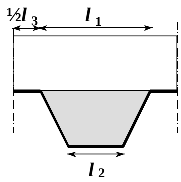
Orientation of parking place:
- Perpendicular to secondary beam

Net height beneath steel beam:2.1 m

Minimum size of secondary beam cross section	Standard level		IPE240
	last level		IPE270
Minimum size of main beam cross	Standard level		IPE400
	last level		IPE450
Design of column cross-section	Carbon steel grade	Available of section type	HEA, HEB or HEM
		Maximum load level (**)	0.35
	Stainless steel grade EN1.4404	Available of section type	CHS or RHS
		Maximum load level (**)	0,45
Requirement to be applied to concrete slab	Total depth of slab		? 120 mm and ? 140 mm
	Maximum height of steel deck		62 mm
	Minimum compactness of rib of steel deck (*)		0.393
	Minimum thickness of steel sheet		0,75 mm
	Minimum mesh of reinforcing steel		ϕ7 150x150 mm
	Location of reinforcing steel mesh		30 mm from top of slab

(*)compactness of rib of steel deck:

$$\frac{(\ell_1+\ell_2)}{2(\ell_1+\ell_3)}$$



(**) Load level: ratio of applied load under fire situation over ultimate load at room temperature design

Table 6: Example of design table for open car parks with a steel and concrete composite structure frame of 7.5 m x 7.5 m

6. GENERAL CONCLUSIONS AND RECOMMANDATIONS FOR FURTHER WORK

Simple calculation rules based on the corresponding numerical results and the fire design approach of Eurocode 3 have been proposed to check the buckling resistance of external SHS column with stainless steel. It has been shown through the comparison with numerical results that the proposed simple calculation rules are able to predict with a quite good accuracy the fire resistance of external stainless steel column under compression. Nevertheless, all these results are only based on advanced calculation tools without any experimental investigation on fire behaviour of loaded external steel members. So some fire tests could be initiated in order to bring confident evidence about the fire resistance of external stainless steel members.

Moreover, numerical analyses performed on standard open car parks subject to natural fires had shown that hollow stainless steel columns filled with concrete can be used without any protection where maximum load level of column is lower than 0.45.

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