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Fire safety engineering — Performance of structures in fire —

Part 6: Example of an eight-storey office concrete building

Ingénierie de la sécurité incendie — Performance des structures en situation d'incendie —

Partie 6: Exemple d'un immeuble de bureaux de huit étages en béton renforcé

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Foreword

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The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

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This document was prepared by Technical Committee ISO/TC 92, *Fire safety*, Subcommittee SC 4, *Fire safety engineering*.

Introduction

The work described in this document is an example of the application of ISO 24679-1. The procedure in this document is intended to follow the principles outlined in ISO 24679-1. The sections of ISO 24679-1 which are considered relevant to this example are identified and thus, the section titles are the same and appear in the same order.

The purpose of this study is to demonstrate the application of the steps outlined in ISO 24679-1 for fire safety engineering and performance of structures in fire in compliance with the related standards of France. As such, the relevant sections to this example are applied and discussed.

Fire safety engineering — Performance of structures in fire —

Part 6: Example of an eight-storey office concrete building

1 Scope

This document provides an example of fire safety engineering design in the application of ISO 24679-1 to an office building.

In this document, an overall structural analysis of a building is undertaken. It consists in a numerical assessment of the structural performance of an eight-storey concrete building when subjected to a fire. This analysis is performed in order to demonstrate that the fire safety objectives, for the relevant design fire scenarios, due to structural behaviour of building in the event of fire, are met with the trial plan for the safety of structure. With regards to this, a fully developed fire was studied.

The purpose of this document is to assess the performance of an office building which is fully accessible to public in case of fire, using ISO 24679-1. In this respect, a critical design fire was identified and analysed using detailed fire modelling. A more detailed analysis was then performed for critical design fire using the finite element model. The advanced model provided all the comprehensive information necessary for analysing the given built environment with respect to fire safety.

It is to be noted that this document only addresses the fire safety objectives related to the structural performance during fire. The analysis within this document is therefore only part of the overall building fire safety strategy.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 24679-1:—[1)], *Fire safety engineering — Performance of structures in fire — Part 1: General*

3 Terms, definitions and symbols

3.1 Terms and definitions

For the purposes of this document, the terms and definitions given in ISO 24679-1 apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— IEC Electropedia: available at http://www.electropedia.org/

— ISO Online browsing platform: available at http://www.iso.org/obp

1) Under preparation. Stage at the time of publication: ISO/DIS 24679-1:2017.

3.2 Symbols

A_f	floor surface area
A_t	total area of enclosure (walls, ceiling and floor, including openings) (m^2)
A_v	total area of vertical openings on all walls (m^2)
D	diameter of fire source (m)
EC	Eurocode
O	opening factor of the fire compartment ($m^{1/2}$)
O_{lim}	reduced opening factor in case of fuel controlled fire ($m^{1/2}$)
RMT	maximum rebar temperature (°C)
T_g	gas temperature (°C)
T_0	ambient temperature (°C)
$V_{Ed,fi}$	the design value of the fire induced shear load
$V_{Rd,fi}$	the design value of shear resistance in case of fire
b	thermal inertia for the total enclosure ($J/m^2{\cdot}s^{1/2}{\cdot}K$)
c	specific heat (J/kg·K)
dx	grid size (m)
h_c	convective coefficient of exposed side ($W/m^2{\cdot}K$)
h_b	convective coefficient of unexposed side ($W/m^2{\cdot}K$)
h_{eq}	weighted average of window heights on all walls
k	thermal conductivity (W/m·K)
m	combustion factor
$q_{f,d}$	design fire load density related to the floor area A_f (MJ/m^2)
$q_{f,k}$	characteristic fire load density related to the surface area A_f (MJ/m^2)
$q_{t,d}$	design fire load density related to the surface area A_t (MJ/m^2)
t_{lim}	time to reach maximum gas temperature in case of fuel controlled fire (h)
t_{max}	time to reach maximum gas temperature (h)
t_{RMT}	time to reach the rebar maximum temperature (min)
Γ	time factor function of the opening factor O and the thermal absorptivity b
Γ_{lim}	time factor function of the opening factor O_{lim} and the thermal absorptivity b
ε	surface emissivity of the member
ρ	density (kg/m^3)

δ_{q1} activation risk due to the size of the compartment equal to 1 in this example

δ_{q2} fire activation risk due to the type of occupancy equal to 1 in this example

δ_n factor taking into account different active fire-fighting measures equal to 1 in this example

4 Design strategy for fire safety of structure

4.1 General design process for fire safety of structures

This example studies the fire resistance assessment of an office building using ISO 24679-1. According to the history of the real fires in an open plan office, a generalized fire is possible.

Potential design fire scenarios in the build environment were studies. Temperature time curves were produced and analyses were carried out so as to identify the plausible worst case scenarios[1], [2].

In order to provide a more detailed and broader resolution of the fire scenario for the use of detailed structural analysis, the critical design fire scenario was investigated using advanced fire modelling and design fire was established.

The comprehensive structural behaviour was studied via advanced structural modelling of the worst case scenario.

Additionally, factors and influences in quantification process and uncertainty of material properties where studied. As such, a more detailed analysis was carried out by means of sensitivity analyses in which the OAT (i.e. one-factor-at-a-time) was used where only one input variable in the base case fire scenario was changed[3]. In this respect, a range of input variables in generating the fully developed fire and heat transfer models were investigated by means of a literature study.

4.2 Guidance of practical design process for fire safety of structure

ISO 24679-1:—, Table 1, illustrates various steps and parameters to be considered when assessing the behaviour of structures subjected to fire exposure. The details of the relevant steps to this example are presented in the following clauses.

5 Qualification of the performance of structures in fire

5.1 STEP 1: Scope of the project for fire safety of structure

5.1.1 Built-environment characteristics

The building studied is an open-plan office building without any interior vertical compartmentations, with a glazed façade all around the perimeter. It has a floor area of approximately 420 m^2 and total gross area of 3 360 m^2. The building is divided into two basement levels, a ground floor, and five floors above ground, which are open to the public. The building is 30,25 m long by 14,25 m wide and 25 m high. The ground floor has a height of 4 m whereas the upper storeys have a height of 3 m. Elevators and staircases are placed in the central core.

The length is divided into five structural bays, and the width into two bays. Each bay is 6 m by 7,125 m as shown in Figure 1. The building frame is composed of reinforced continuous concrete beams and columns, supporting concrete floor slabs which are 180 mm thick; the exterior walls are 200 mm thick; the columns are 500 mm by 500 mm wide, and the beams are 400 mm deep by 250 mm wide.

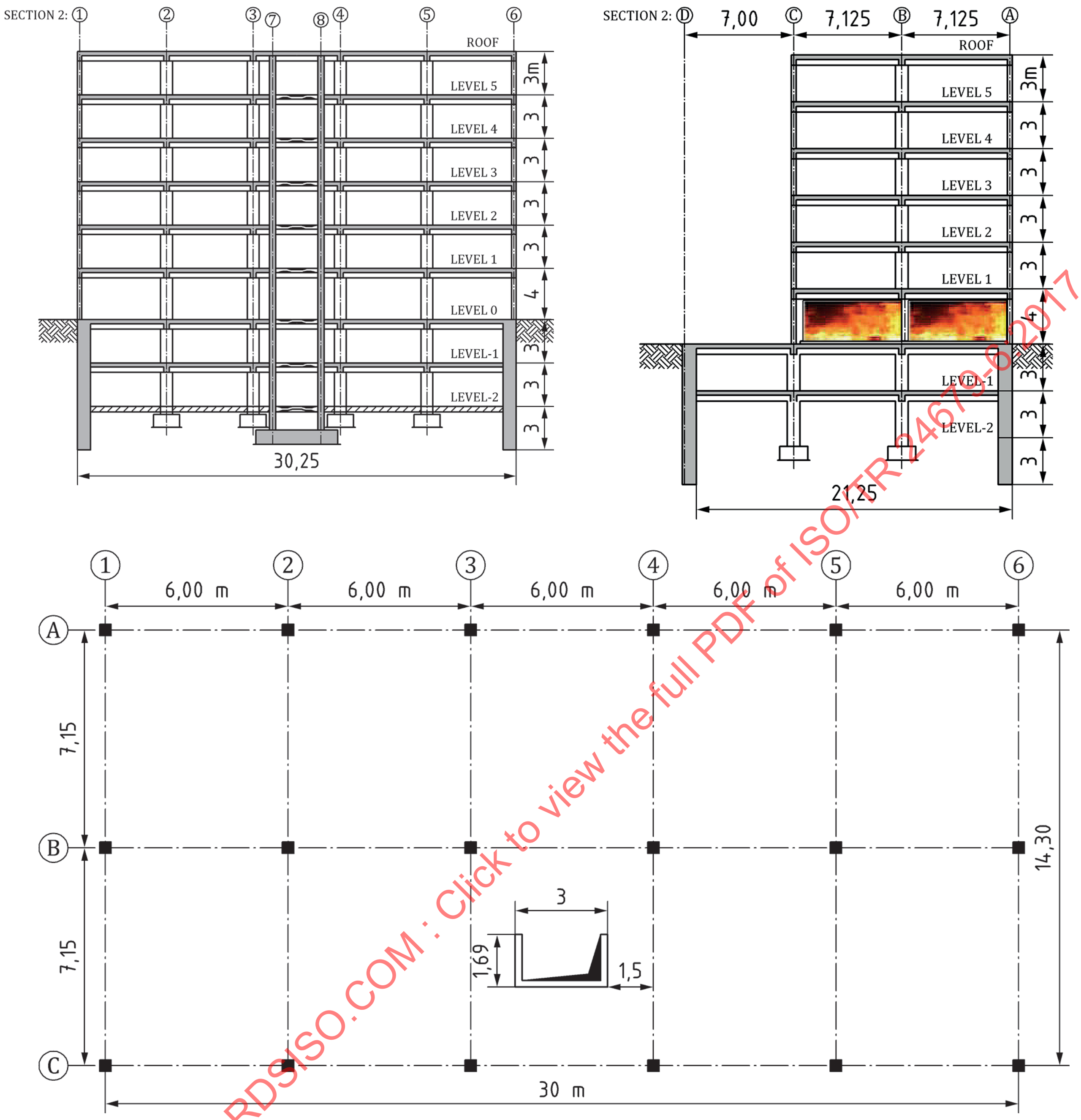


Figure 1 — Plan and elevation of the structure, dimensions in metres

The structure includes three kinds of structural members: reinforced concrete columns, beams and slabs. The cross section of the column is equal to 0,25 m^2 and is presented in Figure 2. For the first floor, the height of the column is equal to 4 m whereas the upper storeys have a column height of 3 m. The materials are:

— concrete: C30/37;

NOTE 30 and 37 are the characteristic cylinder and cube compressive strengths, respectively, in Mpa.

— steel: hot rolled, grade 500 class B.

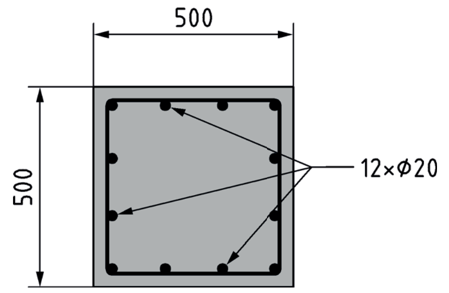


Figure 2 — Column cross section

The reinforcement in the column and the axis distance are presented in Table 1.

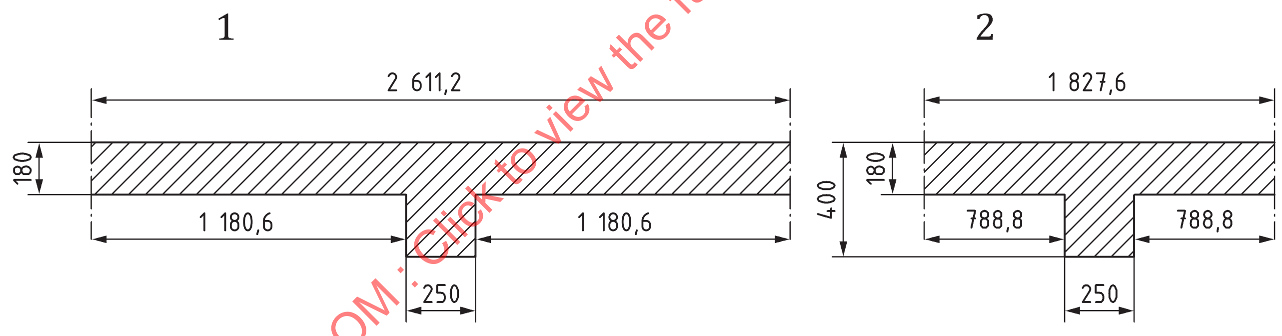
Table 1 — Column reinforcements and the axis distance of reinforcements

Column	Ø	Axis distance
Longitudinal reinforcement	12 Ø 20	52 mm
Stirrups	Ø12/200 mm	36 mm

In Figure 3, the cross sections of the beams are illustrated. The materials are as follows:

— concrete: C25/30;

— steel: hot rolled, grade 500 class B.



Key

1 cross-section at mid-span

2 cross section at intermediate support

Figure 3 — Continuous beam cross section

The reinforcement and the axis distances in the beams are presented in Table 2.

Table 2 — Reinforcement and axis distance in the beams

Beam	Perimeter support	Mid-span	Intermediate support	Axis distance
Upper	7 Ø12	2 Ø10	9 Ø12	42 mm
Lower	3 Ø16	3 Ø16	3 Ø16	44 mm
Stirrups	Ø8/175 mm	Ø8/175 mm	Ø8/175 mm	34 mm

The slab is 180 mm thick and the reinforcement is presented in Figure 4 and Table 3. The materials are as follows:

— concrete: C25/30 concrete;

— steel: hot rolled, grade 500 class B.

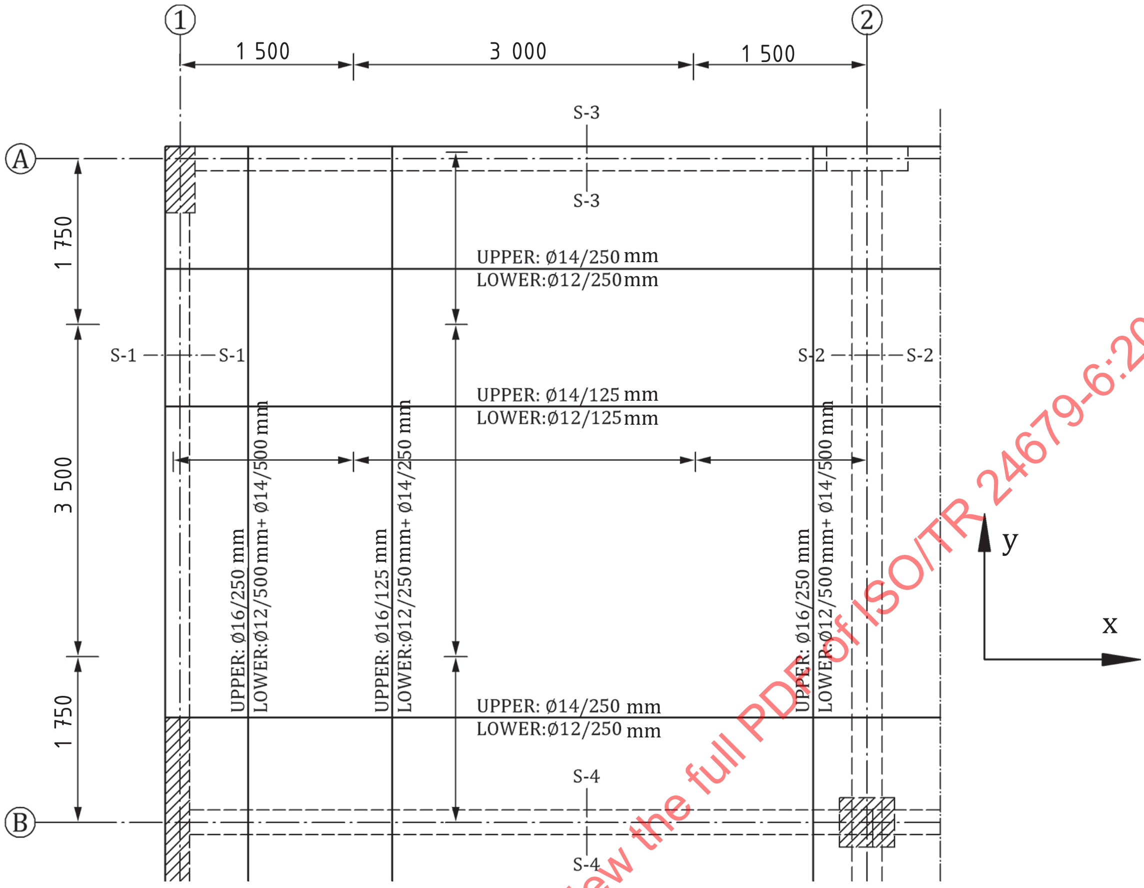


Figure 4 — Reinforcement distribution in the slab

Table 3 — Slab reinforcement and axis distance

X direction slab	**Middle strip (3 m)**	**Axis distance**
Upper	Ø14/125 mm	37 mm
Lower	Ø12/125 mm	36 mm
Y direction slab	**Middle strip (3 m)**	**Axis distance**
Upper	Ø16/125 mm	52 mm
Lower	Ø12/250 mm Ø14/250 mm	49 mm

NOTE Stress-strain relationships of concrete and steel are given in Reference [4] As such, an explicit model for transient strain was applied. Tensile strength of concrete has been considered in the advanced modelling.

5.1.2 Fuel loads

The building is an office space with cellulosic (i.e. majority of fuel load), plastic, and miscellaneous type fuel, which is assumed to be uniformly distributed throughout the compartment. The fuel load varies greatly depending on the building types and available guidance provides typical ranges. Moreover, the fire load density from construction elements, linings and finishing should be calculated and added to the tabulated fire load densities[5][6].

There is no clear information on the use of this building. As such, potential fire scenarios with possible fire load densities for sparsely furnished (i.e. office machine sales) to densely loaded (i.e. business office, library) ranging from 350 MJ/m^2 to 1 350 MJ/m^2[6] were examined to define the acceptable fuel load density in this building.

5.1.3 Mechanical actions

Dead and live loads are presented in Table 4.

Table 4 — Loads

	Load name	Value of load
Dead load	Self-weight	25 kN/m^3
	Finishing, pavement, embedded services, partition	1,5 kN/m^2
Live/variable load	Office	4 kN/m^2

The mechanical action in fire situation was determined in accordance with Reference [7]. Consequently, the load combination given in Formula (1) was used:

$$G + \Psi_{2,i} Q_i \tag{1}$$

where

G is the sum of all the permanents loads;

Q is the dominant live load;

Ψ_2 = 0,6.

The loads which were used for calculation with an advanced structural model (finite element model in 3D that takes account of the nonlinearities of materials and geometric) are summarized in Table 5.

NOTE No wind action was considered because Ψ_2 = 0 for wind. The charge from upper storeys will be added in detailed structural modelling.

Table 5 — Used loads in detailed structural analysis

				Unit
Slab	Dead load (G)	0,18 × 25 +1,5	6	$\frac{\text{kN}}{\text{m}^2}$
	Live load (Q)	4	4	
	Total load	**G + 0,6Q**	**8,4**	
Exterior beams	Dead load (G)	0,25 × 0,40 × 25	2,5	$\frac{\text{kN}}{\text{m}}$
	Dead load façade (G_F)	8	8	
	Total load	**$G+G_F$**	**10,5**	
Interior beams	Dead load (G)	0,25 × 0,40 × 25	2,5	$\frac{\text{kN}}{\text{m}}$
	Total load	**G**	**2,5**	

For the detailed structural analysis, only the first floor was modelled in a structural modelling software. This assumption was discussed in 5.6.2. The action of the floors above was considered in the detailed structural analysis, by applying vertical forces acting at the top of the first floor columns.

The vertical forces representing the floors above are presented in the Figure 5.

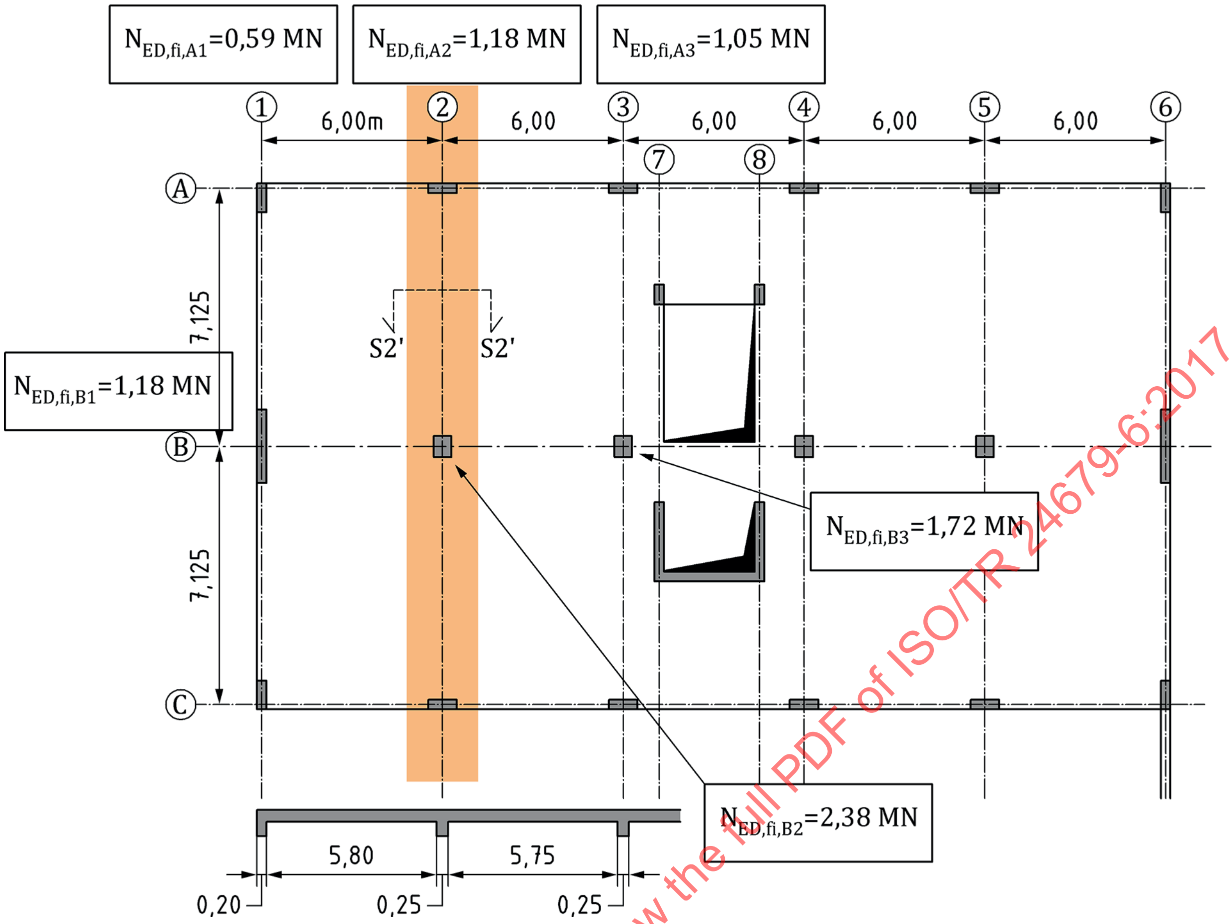


Figure 5 — Vertical forces representing the floors above

5.2 STEP 2: Identifying objectives, functional requirements, and performance criteria for fire safety of structure

The objectives of structural fire safety in this study are:

— life safety of occupants, fire-fighters and others in the vicinity of building in terms of structural behaviour of building in the event of fire;

— conservation of property and continuity of operation.

The functional requirement consists of structural stability to prevent failure of any structural element during the entire duration of fire, including the cooling phase. It results to prevent the fire from spreading to other storeys due to failure of floor and ceiling within the compartment.

A set of performance criteria were selected to fulfil the above objectives and functional requirements:

— the maximum temperature in the tension reinforcement of the concrete structure: It allowed to rapidly compare the relative impact of large numbers of potential design fire scenarios and to identify the critical fire scenario for detailed structural analysis[8].

The following performance criteria in terms of structural stability were assumed for the detailed structural analysis:

— no overall failure of the building, e.g. due to the loss of stability of columns, shear failure, rotational capacity;

— maximum deflection of all slabs does not exceed 1/20 of their spans;

— rotational capacity does not exceed 250 mrad[9].

The critical temperature of tension reinforcement at elevated temperature was calculated assuming a reduction of 0,6 for the design load level in the relevant the fire situation[4] and a partial safety factor of 1,15 for reinforcing steel according with Reference [10]. As such, failure in selected structural member occurred when the characteristic strength of reinforcement steel reached 0,52 of its original capacity. In accordance with 4.2[4], for reinforcing steel (hot rolled) in concrete and for strain larger than 2 % (which is the case for slabs and beams with no high reinforcement ratio), the critical temperature is 583 °C. For the purpose of this document, the value was conservatively taken as 560 °C.

Figure 6 shows a summary of applied methodology in this document.

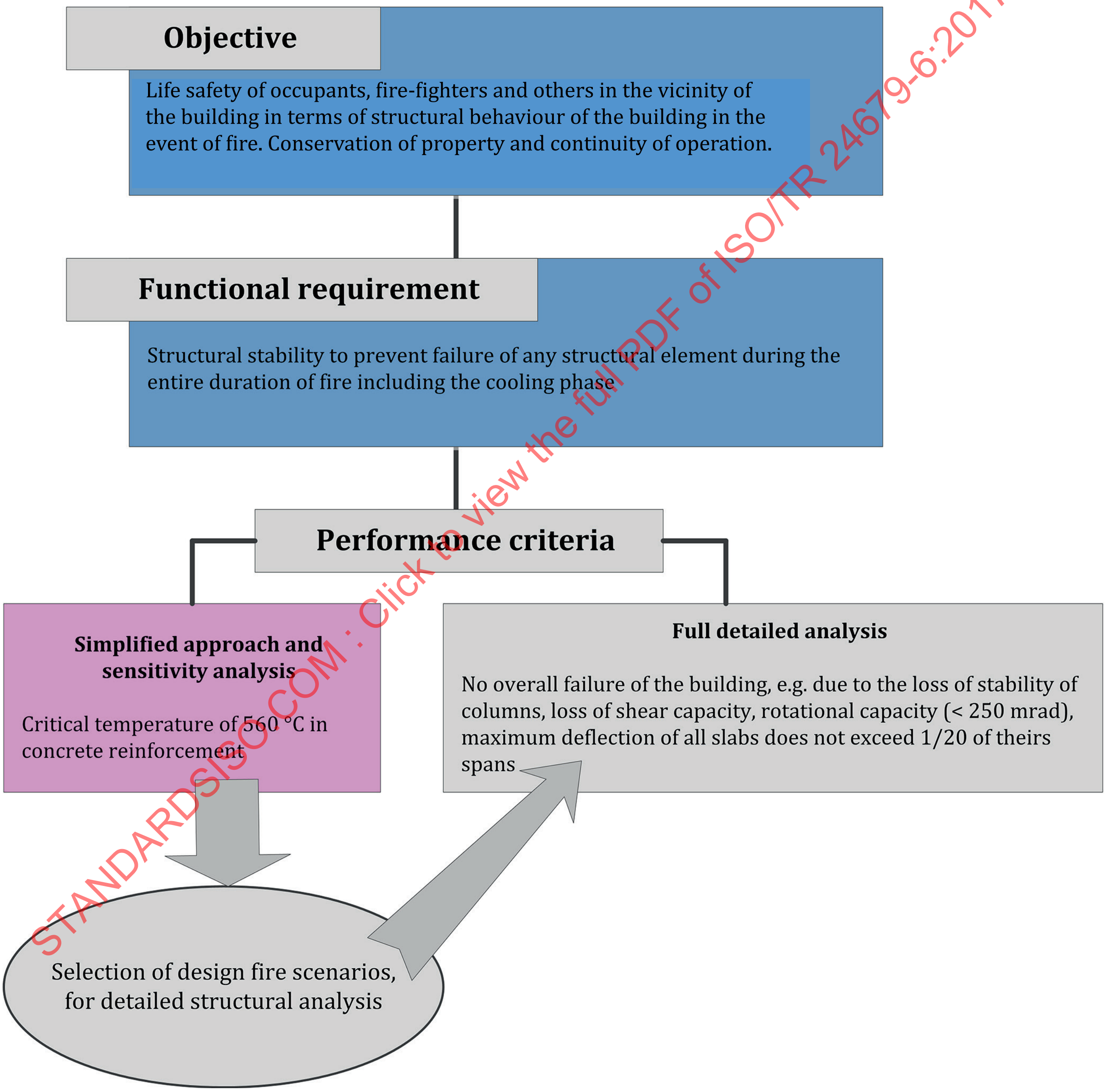


Figure 6 — Schematic overview of the applied methodology

5.3 STEP 3: Trial plan for fire safety of structure

The office building studied here has been designed for the ambient temperature. Structural elements are not protected by any passive or active fire protection systems.

The staircase with surrounding walls of 120 mm has a fire resistance of R120 in accordance with References [4] and [11], which provides the safe egress route for the people and firefighters in the vicinity of building during the fire. Lift should not be used in the case of the fire.

It was assumed that floors and roof sustain their structural fire performance over the whole duration of fire and does not contribute to interior fire spread. This was examined in the detailed structural analysis in the following clauses. The external fire spread to other storeys is assumed to be controlled by measures such as fire protection glass. As such, no multi-storey fire was considered in this example.

5.4 STEP 4: Design fire scenarios and design fires

5.4.1 General

Design fire scenarios and design fires are an important step in the assessment of the performance of structures in fire. It should be noted that a design fire scenario is a specific qualitative description of the development of a fire whereas a design fire (thermal action) is a quantitative description of assumed fire characteristics within a design fire scenario.

See ISO 16733-1[2] and ISO/TS 16733-2[12] for more information about the selection of design fire scenarios and design fires.

5.4.2 Design fire scenarios

The relative impact of a fully developed fire was studied according to the history of the real fires in an office. The location of fire was taken to occur on the ground floor. This enabled the structural effects of a low level story fire to be analysed without the need to explicitly consider the effects caused by the foundations or the top storey of the building. This also enables to determine the performance of structure considering that columns in the ground floor were the most critical load bearing elements and had the largest height in the studied building. Nevertheless, it seems obvious that the size of the concrete columns and their reinforcement ratio are varying over the various levels of the building, but for the purpose of simplicity of this example, it is assumed that the fire in the ground floor is the best representative fire scenario. All stories have an identical architectural plan with identical fire scenarios.

This building is an open-plan office without any interior compartmentation and a fully developed fire engulfs the whole floor surface.

The design Fire scenario was selected as follows.

Initially, a group of fire scenarios were produced using a fuel load density of 511 MJ/m^2, 80 % fractile) in accordance with Reference [5] and different opening factors (ventilation sizes). The worst case scenario was identified as the “base case scenario”. The base case was then used to produce other potential scenarios for the possible range of fuel load. Potential fire scenarios were compared to identify the acceptable fuel load density in this building and therefore the plausible worst fire scenario. It was then examined whether changes in opening factors have any impact on the identified fire scenario.

At this stage, the temperature in the reinforcement rebar in concrete element was used as a simple and easily comparable scale to compare the relative impact of different potential fire scenarios and to identify the worst condition.

5.4.3 Design fires

5.4.3.1 General

At the preliminary stage, the design fires were produced based on the parametric fires[2] and analyses were carried out so as to identify the worst case scenarios. Once previous steps were undertaken, a selection of critical design fires were analysed in more detail using a fire modelling software. The detailed fire modelling results were used as input for the detailed structural analysis.

The method defined in Reference [5] for temperature-time curves of parametric fire, was applied to characterize the fire environment at this stage. An analytical formula that assumes uniformly burning fires (valid for compartment with floor areas up to 500 m² and 4 m height); weighted to take into account the ventilation parameter, fuel load, and the thermal properties of the compartment boundaries. Analytical equation to calculate the parametric-fire curves is presented as follows[5]:

$$T_g = 1\,325[1 - 0{,}324 \exp(-0{,}2t^*) - 0{,}472 \exp(-1{,}7t^*) - 0{,}472 \exp(-19t^*)] \quad (°C) \tag{2}$$

$$t^* = t \cdot \Gamma \quad (h) \tag{3}$$

$$\Gamma = [O/b]^2/(0{,}04/1\,160)^2 \quad (\Gamma \text{ is the time factor}) \tag{4}$$

$$b = (\rho \cdot c \cdot k)^{1/2} \quad (J/\,m^2{\cdot}s^{1/2}{\cdot}K) \quad (\text{where } 100 \leq b \leq 2\,200) \tag{5}$$

$$O = A_v \cdot h_{eq}^{1/2}/A_t \quad (m^{1/2}) \quad (\text{where } 0{,}02 \leq O \leq 0{,}2) \tag{6}$$

The maximum temperature T_{max}, in h, occurs when $t^* = t^*_{max}$, where:

$$t^*_{max} = t_{max} \cdot \Gamma \quad (h)$$

$$t_{max} = \max\,[(0{,}2 \cdot 10^{-3} \cdot q_{t,d}/O);\ t_{lim}] \quad (h) \tag{7}$$

t_{max} is the time to reach maximum gas temperature (h)

$$q_{t,d} = q_{f,d} \cdot A_f/A_t \quad (MJ/m^2) \quad (\text{where } 50 \leq q_{t,d} \leq 1\,000) \tag{8}$$

where $q_{f,d}$ is the design value of fire load density related to the surface area, A_f. In case of slow fire growth rate, t_{lim} = 25 min; in case of medium and fast growth rate, t_{lim} = 20 min and 15 min, respectively:

$$q_{f,d} = q_{f,k} \cdot m \cdot \delta_{q1} \cdot \delta_{q2} \cdot \delta_n \tag{9}$$

where

$q_{f,k}$ is the characteristic fire load density per unit area, in MJ/m²;

m is the combustion factor;

δ_{q1} is the fire activation risk due to the size of the compartment;

δ_{q2} is the fire activation risk due to the type of occupancy;

δ_n is a factor taking into account different active fire-fighting measures.

In case of fuel-controlled fire where $t_{max} = t_{lim}$, then:

$$t^*_{max} = t_{lim} \cdot \Gamma_{lim} \text{ (in h)} \quad (10)$$

$$\Gamma_{lim} = [O_{lim}/b]^2 / (0{,}04/1\,160)^2 \quad (11)$$

$$O_{lim} = 0{,}1 \cdot 10^{-3} \cdot q_{t,d}/t_{lim} \quad (12)$$

where

O_{lim} is the reduced opening factor in case of fuel controlled fire, in $m^{1/2}$;

Γ_{lim} is time factor function of the opening factor O_{lim} and the thermal inertia b.

The temperature-time curve during the cooling phase:

$$T_g = T_{max} - 625\,(t^* - t^*_{max} \cdot x) \qquad \text{for } t^*_{max} \le 0{,}5 \quad (13)$$

$$T_g = T_{max} - 250\,(3 - t^*_{max}) \cdot (t^* - t^*_{max} \cdot x) \qquad \text{for } 0{,}5 \le t^*_{max} \le 2 \quad (14)$$

$$T_g = T_{max} - 250\,(t^* - t^*_{max} \cdot x) \qquad \text{for } 2 \le t^*_{max} \quad (15)$$

where $x = 1$ for $t_{max} > t_{lim}$ and $x = t_{lim} \cdot \Gamma / t^*_{max}$ for $t_{max} = t_{lim}$.

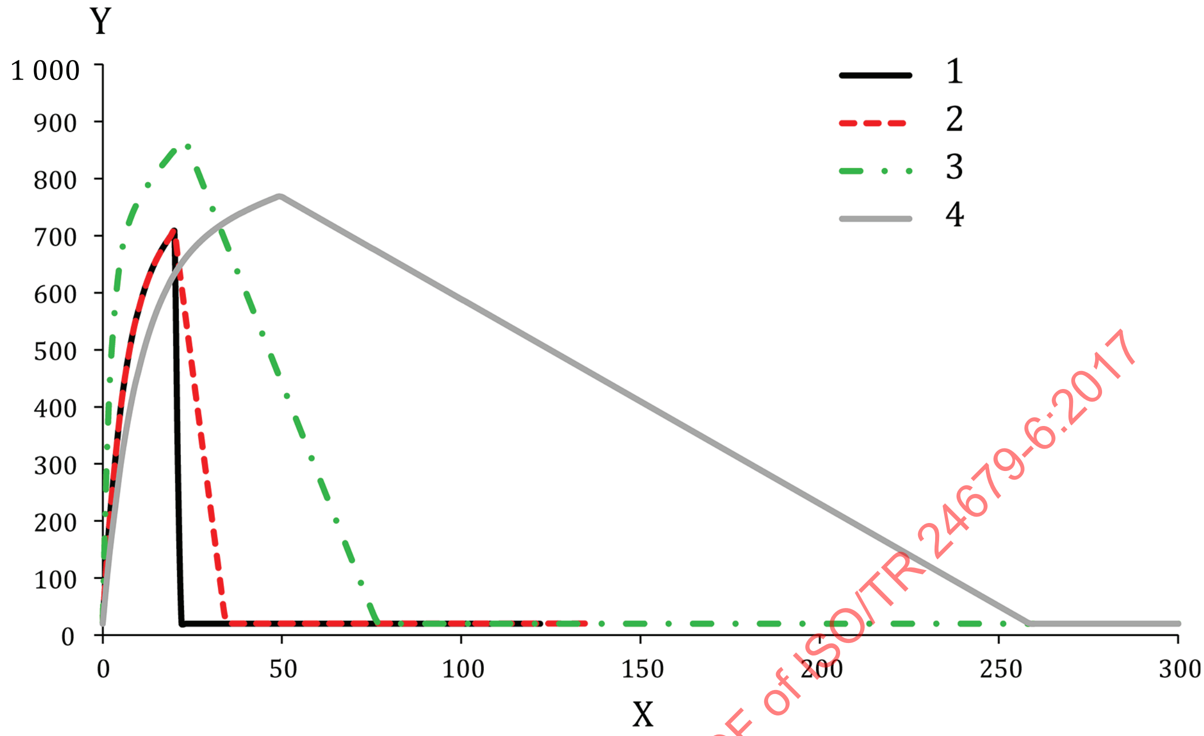
In a fully developed fire, ventilation to the fire compartment and the nature, distribution and quantity of fuel all have a significant effect on duration and severity[13]. In a modern building, the double or triple glazed system may not break as readily as single panels of ordinary glass. Characteristics, orientation and dimension of the glazing façade openings are architectural variable which were not defined in this building. Due to all uncertainties associated with glass breakage and fall-out of glass[14], both fuel-controlled and ventilated-controlled design fires were examined to ensure that the design fire is appropriate to the objectives of the fire-safety engineering analysis as stated[2].

A series of parametric temperature-time curves were produced in which the opening within the façade varied from 100 % to 15 %. The effect of varying opening sizes has been examined based on the limitation imposed by parametric fire methodology for opening factors (i.e. $0{,}02 \le O \le 0{,}2$)[5].

The characteristic fire load density for an office building ($q_{f,k}$) equal to 511 MJ/m^2[5] and the combustion factor of m = 0,8 were taken[5]. δ_{q1}, δ_{q2}, and δ_n are assumed to be equal to 1. As such, size and type of compartment, and fire-fighting measures do not have any reduction impact on the fire load density. The design value of the fire load density ($q_{f,d}$) is therefore equal to 412 MJ/m^2 using Formula (9).

It was assumed that the boundary material of the enclosure consists of normal weight concrete, which has a density of 2 300 kg/m^3, thermal conductivity of 1,33 W/m·K, specific heat of 900 J/kg·K in the ambient temperature in accordance with Reference [4]. These properties are assumed to be constant for the purpose of simplification in this step. In further detailed analysis, these values were temperature dependent.

The calculated gas-phase temperatures are presented in Figure 7.



Key

X temperature, in °C

Y time, in min

1 EC1 – Opening factor = 0,2

2 EC1 – Opening factor = 0,146

3 EC1 – Opening factor = 0,074

4 EC1 – Opening factor = 0,02

Figure 7 — Gas temperatures within the compartment for different opening factors using the EC parametric approach

The results show that opening factors lying between 0,097 $m^{1/2}$ and 0,2 $m^{1/2}$ produces a relatively short fuel-controlled fire. The decrease in opening factor under 0,097 $m^{1/2}$ generates a fire restricted by ventilation. Opening factors between 0,02 % $m^{1/2}$ and 0,074 % $m^{1/2}$ due to the façade glass breakage results in a ventilation-controlled fire with peak gas temperatures lying between 750 °C and 850 °C.

5.4.3.2 Base case design fire

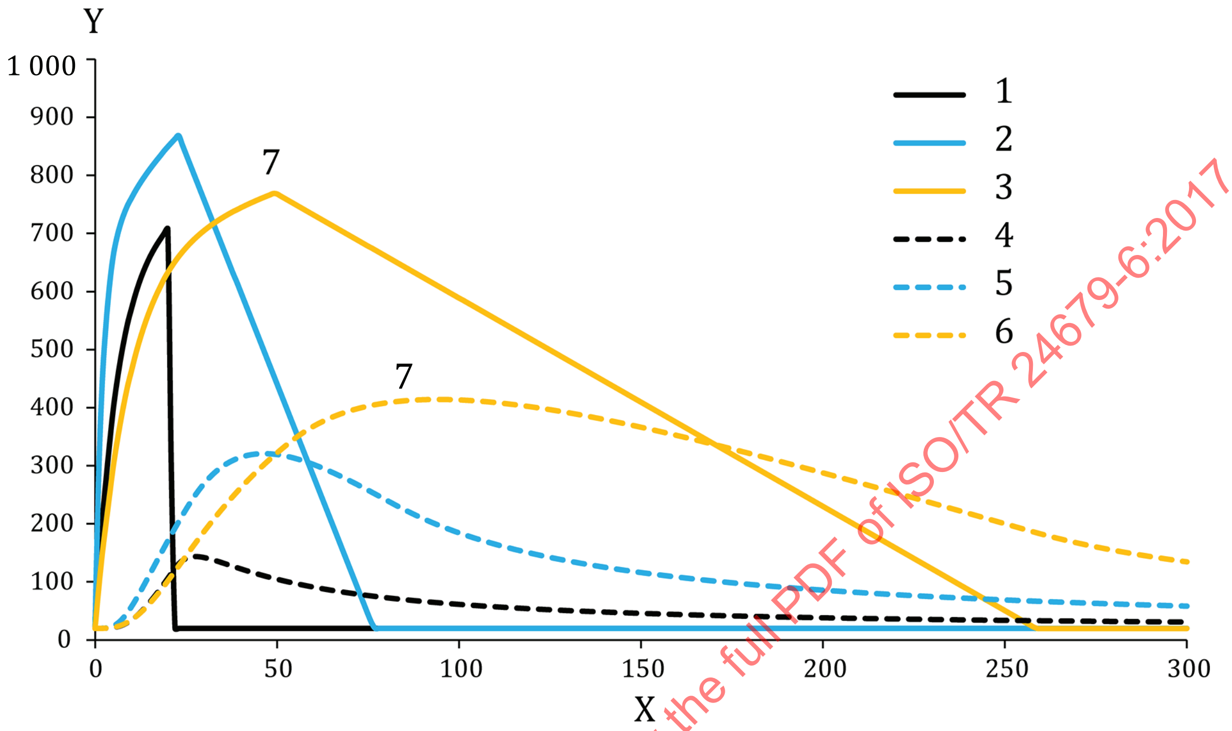
In order to compare and identify the impact of different potential fire scenarios, the above temperature fields were then used as an input to calculate the resulting temperature in the concrete at the location of the reinforcement rebars. The single parameter assessments that are typically used can be applied to comparative assessment[8][15].

One-dimensional conductive heat transfer inside the material was considered alongside boundary conditions for both convective and radiant heating due to the gas temperature[16]. The heat transfer was solved by means of finite differences, as detailed in Annex A.

The simple approach used here allows a large variety of parameters to be calculated rapidly. If a full FEM were to be carried out, this would be computationally restrictive to do.

The concrete properties are assumed to be the same as in the previous section, and are also assumed to be constant. For the purpose of this document, an emissivity of 0,7, and a convective coefficient of 35 $W/m^2{\cdot}K$ for the exposed surface and 4 $W/m^2{\cdot}K$ for the unexposed surface of the concrete element were assumed according to References [4] and [5]. To compare the scenarios, the maximum temperature in the tension reinforcement of the concrete beam at the centre of the rebar (i.e. 44 mm) was calculated.

A comparison of the rebar temperatures induced in the concrete beam with the different thermal actions is illustrated in Figure 8.



Key

X temperature, in °C
Y time, in min
1 EC1 – opening factor = 0,2
2 EC1 – opening factor = 0,074
3 EC1 – opening factor = 0,02
4 rebar temp. – opening factor = 0,2
5 rebar temp. – opening factor = 0,074
6 rebar temp. – opening factor = 0,02
7 base case

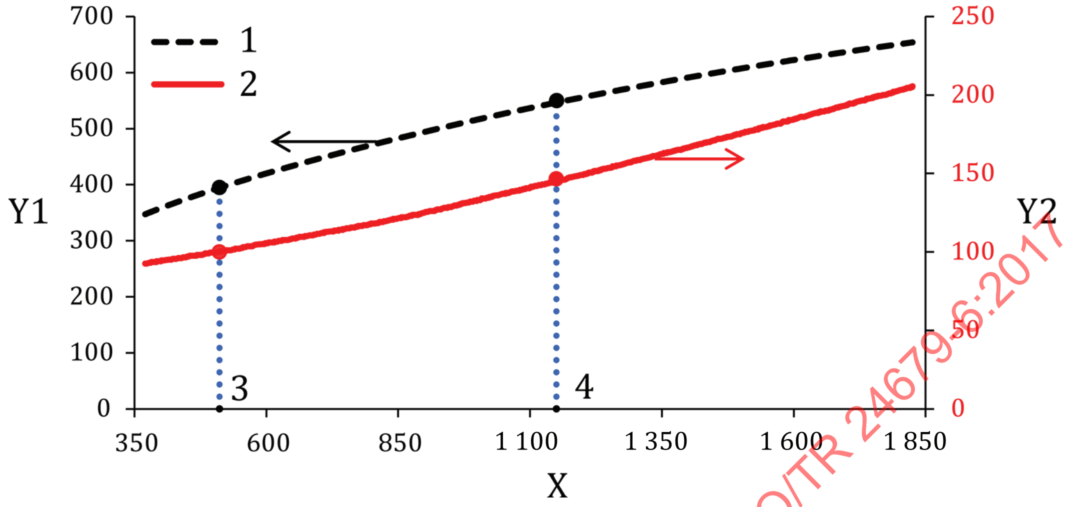
Figure 8 — Parametric temperature-time curves using Eurocode 1 and resulting rebar temperatures in the concrete beam measuring 400 mm × 250 mm

The results indicate that the rebar maximum temperature (RMT) obtained from a parametric temperature-time curve with opening factor 0,097 $m^{1/2}$ (i.e. about less than 15 % façade glass breakage) is approximately 408 °C after 95 min. This was named the “base case” scenario.

5.4.3.3 Potential fire scenarios

The base case scenario with 511 MJ/m^2 fire load density does not threat the structure. The base case scenario was used to produce other potential fire scenarios where the fire load density ranges from 350 MJ/m^2 to 1 350 MJ/m^2 (as discussed in 5.1.2). The relative impact of other potential fire scenarios was compared in order to identify the acceptable fire load density which could endanger this building and consequently to identify the plausible worst design fire scenario.

Figure 9 shows the variations of the peak rebar temperature and the time taken to it relative to the base case with a characteristic fire load density ranging from 347 MJ/m² to 1 350 MJ/m². Denser fuel loads result in higher peak bay rebar temperatures and a longer time to reach the peak value.



Key

X characteristic fire load density ($q_{f,k}$), in MJ/m²

Y1 RMT, in °C

Y2 t_{RMT}, in min

1 rebar max temperature (RMT)

2 time for RMT

3 base case

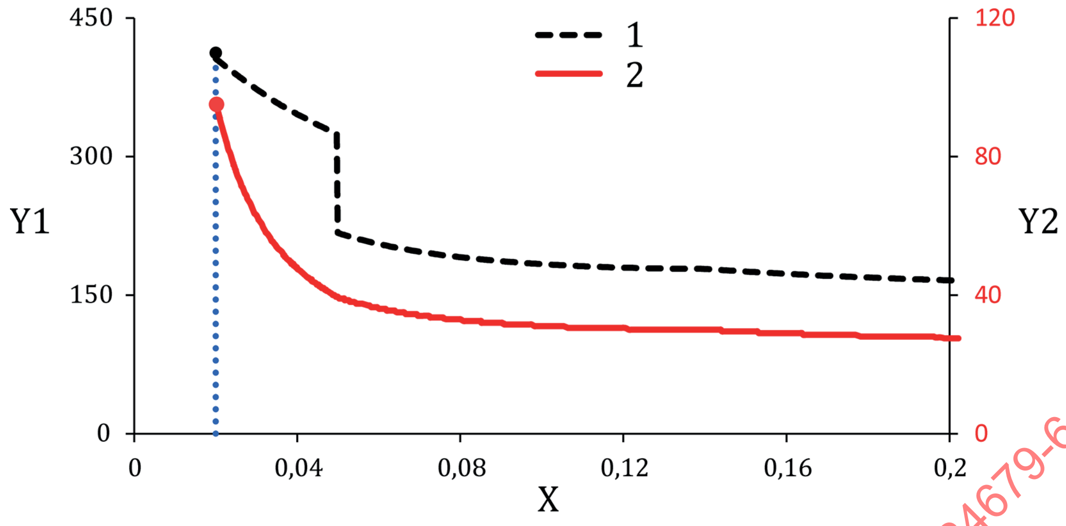
4 critical case

Figure 9 — Peak rebar temperatures and corresponding times vs characteristic fuel load density

The results show that the critical rebar temperature (*critical case*) occurs when the characteristic fire load density is above 1 140 MJ/m² after 140 min, which is the case for densely loaded (i.e. library, business office) spaces.

The acceptable design fuel load density in this building is therefore equal to 912 MJ/m² (i.e. the production of combustion factor of 0,8 and characteristic fire load density) which seems a reasonable value.

The effect of varying opening sizes has been examined where the design fuel load density was 912 MJ/m² and illustrated in Figure 10.



Key

X opening factor, in $m^{1/2}$

Y1 RMT, in °C

Y2 t_{RMT}, in min

1 rebar max temperature (RMT)

2 time for RMT

NOTE The minimum value is obtained from open.

Figure 10 — Peak rebar temperature and corresponding time vs opening factor

The results show that the largest peak value corresponds again to opening factor 0,02 $m^{1/2}$, as was already shown in 5.4.3.2 in the selection of the base case scenario. The gradient change in the peak rebar temperature curve is due to the change in fire environment from ventilation-controlled to fuel-controlled (i.e. at 45 % façade glass breakage and fall-out).

The design fire where the design fuel load is 912 MJ/m^2 for opening factor 0,02 $m^{1/2}$ (15 % opening in the facade) was selected as the critical design fire. The critical fire scenario was then analysed using a zone model and a finite element software to analyse the structural performance.

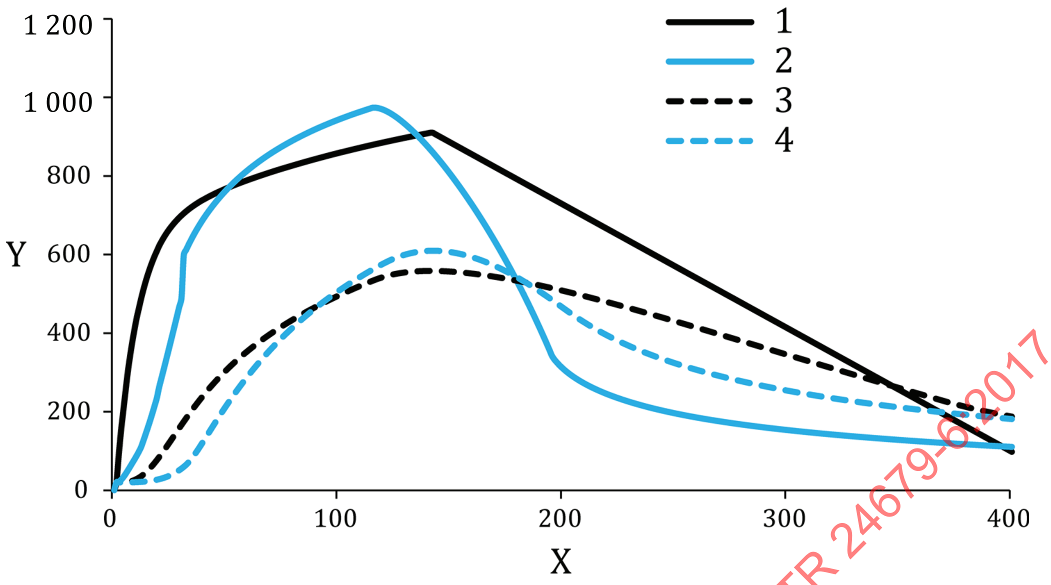
5.4.3.4 Detailed modelling of the critical design fire

This subclause is dedicated to detailed fire modelling using a zone model to evaluate comprehensively the gas temperature to which the structural elements are subjected to. The information provided with numerical model is much more detailed and therefore presents a more comprehensive picture of the thermal actions necessary for a more detailed structural analysis.

The OZone model was used for the purpose of the fully developed fire. It has been validated for different types of fire and compartments[17]. The main assumption in the zone model is that the burning is uniform and the compartments are divided into zones in which the temperature distribution is uniform at any time. In the zone model, used in this document, the temperature is considered to be uniform within the whole compartment and thus is valid for the case of fully developed fires.

The scenario selected was the critical case where the design fuel load was 912 MJ/m^2 (i.e. the characteristic fuel load was 1 140 MJ/m^2) for opening factor 0,02 $m^{1/2}$ obtained from 5.4.3.3.

A comparison of the calculated temperature-time curves using the parametric fire and fire modelling software along with the resulting rebar temperatures is presented in Figure 11 for the scenario selected.



Key

X time, in min

Y temperature, in °C

1 gas temp. critical case eurocode 1

2 gas temp. critical case-ozone

3 rebar temp. – eurocode 1

4 rebar temp. - ozone

Figure 11 — Temperatures within the compartment using the parametric fire approach and fire modelling software for the critical case where the design fuel load was 912 MJ/m² for opening factor 0,02 m$^{1/2}$ in the facade and resulting rebar temperatures in the concrete beam measuring 400 mm × 250 mm

The results show that maximum gas temperature is around 990 °C after 116 min fire exposure and is slightly higher in the OZone model. The results also indicate that the rebar temperature curves follow the same trend.

Therefore, the temperature field of the critical case resulting from fire modelling software was used as an input for the detailed structural analysis.

5.5 STEP 5: Thermal response of the structure

5.5.1 General

In order to analyse the building behaviour, the result of the zone model corresponding to a critical scenario identified in 5.4.3.4, Figure 11, was used.

The following thermal and physical properties for thermal transfer were taken into account as presented in Reference [4]:

— water content: 1,5 % off mass;

— concrete density: 2 300 kg/m³;

— thermal conductivity: lower limit given in Eurocode 2;

— siliceous aggregates;

— emissivity related to the concrete surface: 0,7;

— the coefficient of heat transfer by convection is taken as 35 $W/m^2 \cdot K$, while on the unexposed sides this value is reduced to 4 $W/m^2 \cdot K$.

The thermal actions were applied to different structural members to calculate their heating as a function of time. The heat transfer analysis of all structural members subjected to thermal actions was calculated using a structural analysis software [18].

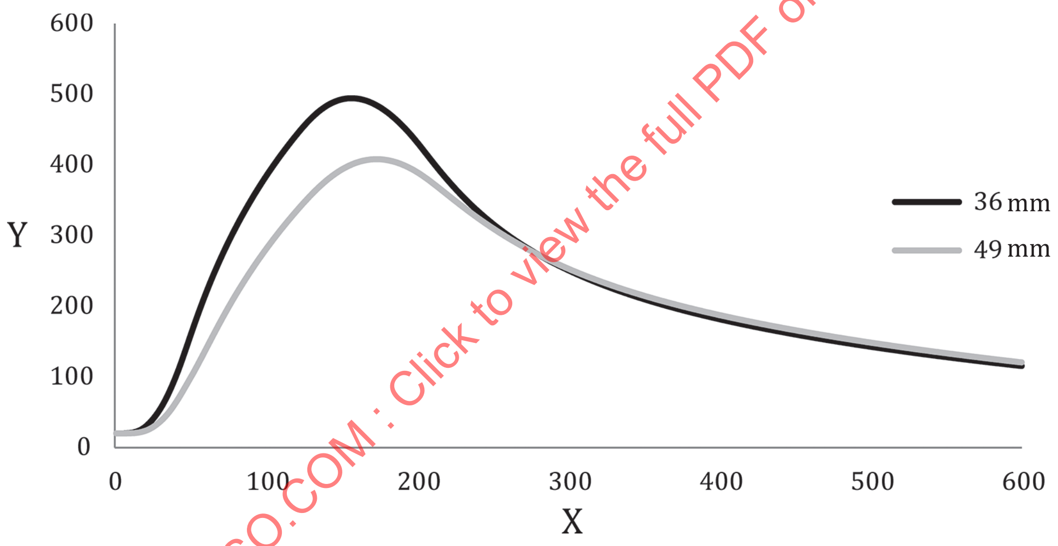
5.5.2 Thermal analysis of the slab

The thermal analysis of the slab cross-section was performed prior to the structural analysis to determine the temperature distribution through the cross-section.

The slab with a thickness of 0,18 m was discretized into 36 quadrilateral elements (5 mm mesh was used).

It was assumed that the temperature of the reinforcement bars was equivalent to the concrete temperature at the same horizontal level in the slab.

The temperature evolution in the bottom reinforcement is presented in Figure 12. The maximum temperature for the x direction reinforcement bars (axis distance 36 mm) is 494 °C and is reached after 156 min of fire exposure. For the y direction reinforcement (axis distance 49 mm), the maximum temperature is reached after 173 min of fire exposure and the value is 408 °C. After reaching the peak value, the temperature decreases.



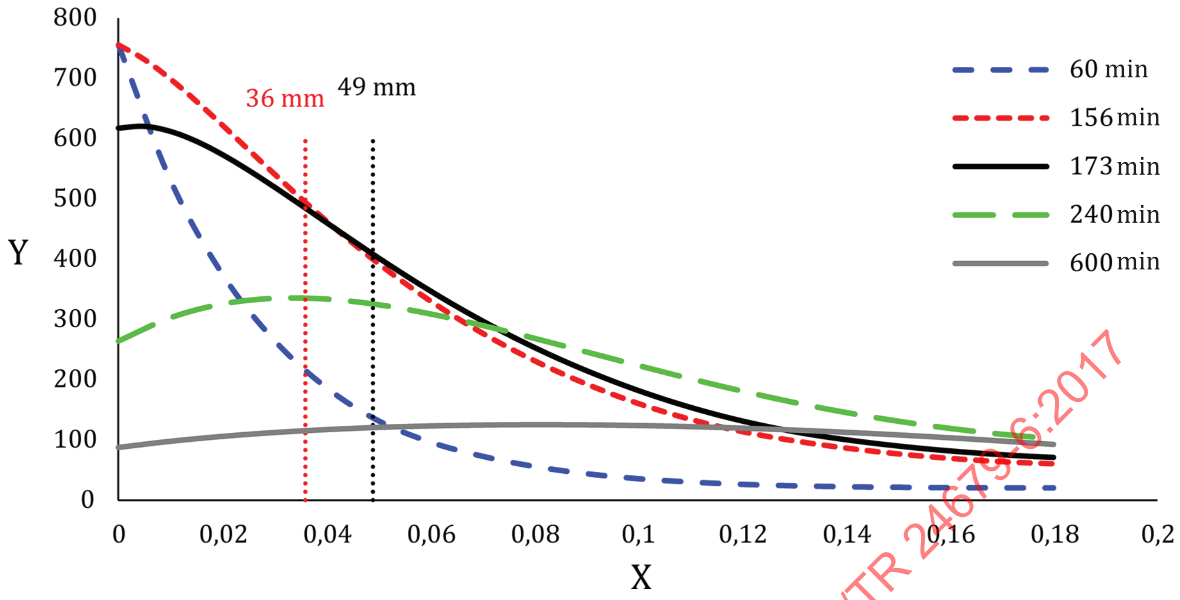
Key

X time, in min

Y temperature, in °C

Figure 12 — Evolution of temperature in the bottom reinforcement

Figure 13 shows the temperature evolution inside the slab for every hour during the fire exposure.



Key

X depth, in m

Y temperature, in °C

Figure 13 — Temperature evolution inside the slab

5.5.3 Thermal analysis of the beam

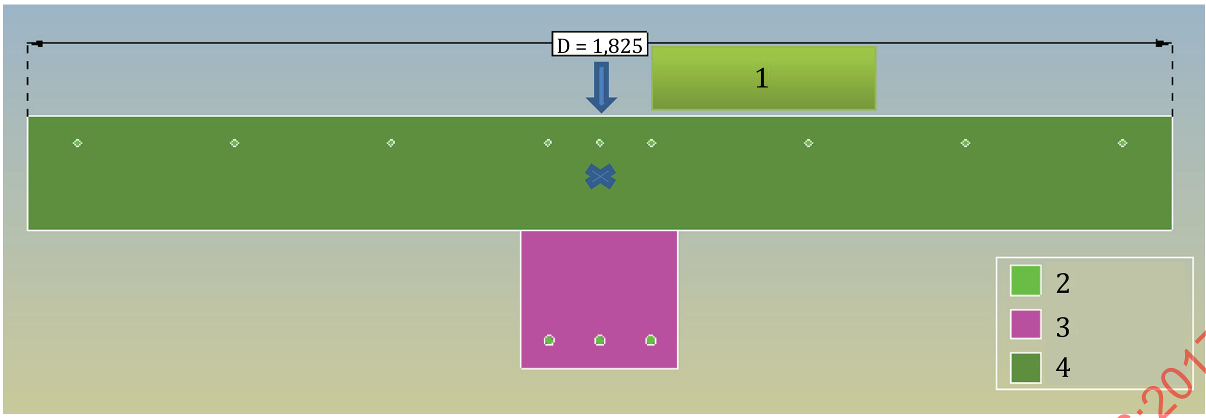
The beam thermal model is presented in Figure 14.

In order to build the finite element modelling of the structure, beams and shell elements were considered. As far as a global analysis was performed and connections were taken into account, some finite model elements of shell and beam need to be superposed to represent the slabs and the beams of the structure. However, in order to obtain the right structural rigidity and to have the right representation of the thermal field, the beam was modelled in two parts:

— the upper part of the beam (superposed with the shell element representing the slab) was assumed to consist of non-load bearing material which did not contribute to the strength and stiffness of the beam in the structural analysis ;

— the bottom part of the beam is in concrete.

To ensure connection with the shell elements, the node-line of the beam elements was located at mid-level in the slab.



Key

1 beam nodeline

2 steel

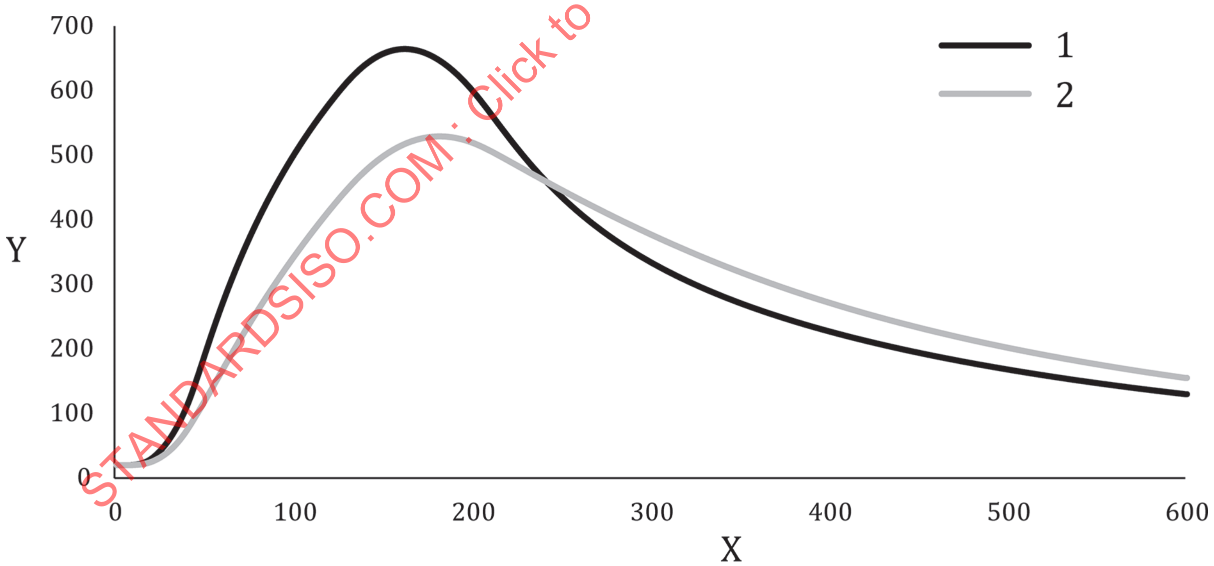
3 concrete

4 concrete

Figure 14 — Thermal model of the beam

The evolution of temperatures in the reinforcement rebars were presented for the interior beams. The temperatures were presented for the bottom reinforcements.

For the bottom corner reinforcement rebars, the peak temperature of 665 °C is reached after 162 min. After 10 h of fire exposure, the temperature decreases and the value was 130 °C. For the bottom middle reinforcement bar, a temperature of 529 °C is reached after 181 min. After 10 h of fire exposure, the temperature decreases to 155 °C (see Figure 15).



Key

X time, in min

Y temperature, in °C

1 corner re-bars

2 middle re-bars

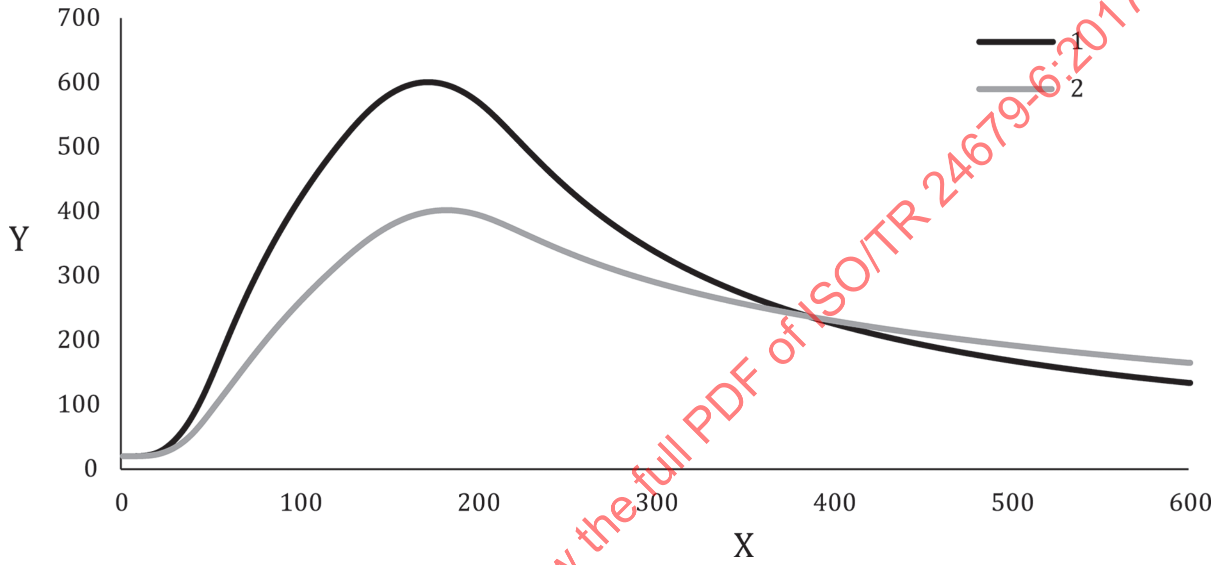
Figure 15 — Temperature-time curves for bottom reinforcement bars

5.5.4 Thermal analysis of the column

The node line of the column was considered at the centre of the section. For the edge columns, only three faces were exposed to fire while for the corner column, only two faces were exposed.

The rebar temperature evolutions in time are presented for a central column (see Figure 16). After 172 min, a peak temperature of 601 °C is reached in the corners reinforcement. 10 min later (i.e. after 182 min), the middle reinforcement bars reach their peak temperature of 402 °C. After 10 h of exposure, the rebar temperature was below 200 °C.

The evolution of temperature in time can be observed in Figure C.1.



Key

X time, in min

Y temperature, in °C

1 corner re-bars

2 middle re-bars

Figure 16 — Column rebars temperature in time

5.6 STEP 6: Mechanical response of the structure

5.6.1 Structural model

For the structural model in SAFIR, only the first floor was considered while the floors above were represented by vertical forces acting at the top part of the column. Because the building is symmetric, only half of the first floor was represented.

The column and beam members were discretized in beam elements with a length between 0,25 m to 0,5 m.

The concrete slab was modelled as rectangular shell elements (1 124 shell elements in total), with sizes between 0,25 m to 0,5 m side.

In the structural analysis, the node line was defined for all the elements. The elements were connected through the node line, the loads are applied at this level, and the results are plotted for the level of node line level. The node line of the shell elements was at mid-depth of the slab, while the node line of the columns was at the centroid of the cross section. The node line of the beam is presented in Figure 14.

A typical view of the structural model and boundary conditions are presented in Figure B.1. The columns were fully fixed at the bottom. For the slab, the rotation after y- and z-axes was restrained at the symmetry line, as well as the displacements along the x-axis.

In Figure 17, the applied loads on the structure for the beams, columns and shell elements are shown.

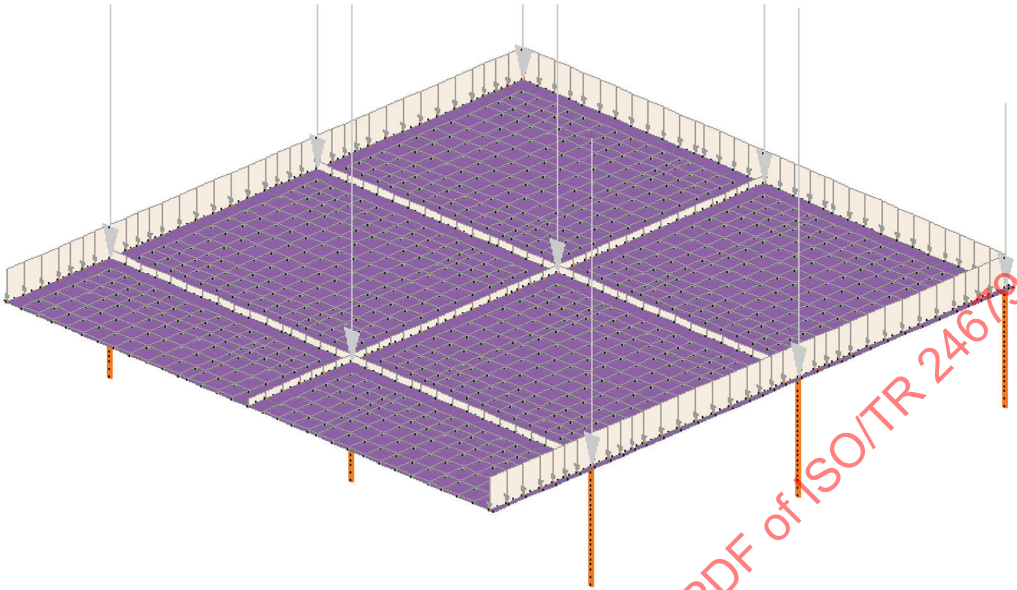


Figure 17 — Applied loads on the structure (distribution loads on the shell and beam elements and vertical forces on the top of the columns)

The geometric properties of the beams and shell elements are presented in Figure B.2 and Figure B.3.

5.6.2 Assumptions of the analysis

5.6.2.1 SAFIR beam elements

A number of assumptions have been made for the beam elements:

a) the Bernoulli hypothesis was considered;

b) shear energy of the plane sections in the finite elements was ignored;

c) plastifications were only considered in the longitudinal direction of beam elements, this means uniaxial constitutive models were used in the beam element;

d) non-uniform torsion was accounted for, but the stiffness in torsion remains constant during the analysis;

e) large displacements were considered but strains were assumed to be small.

5.6.2.2 SAFIR shell elements

There were a number of assumptions for the shell elements:

a) the shear energy in the finite elements was neglected;

b) the out-of-plane displacements and rotations were parabolic along each side;

c) the rotations along the edges varied linearly;

d) the shell elements had a cubic membrane displacement field;

e) the cross section of reinforcement bars was not subtracted from the plane section of shell elements; steel and concrete were simultaneously present at the location of reinforcement bars;

f) only axial direction action was resisted by the reinforcement bars; the reinforcement bars could not directly resist shear forces.

5.6.2.3 General assumptions of the analysis

a) The fire exposure beneath the slab was uniform.

b) No holes due to the stairs and elevator in the slabs were considered.

c) The same cross section and reinforcement ratio was used for the exterior beams and the interior ones.

d) All the columns used in the analysis had the same cross section and reinforcement ratio.

e) Finite element model does not consider shear failure. Specific checks could be done following Reference [4], Annex D3 recommendation, but it was not the aim of this document.

f) For the columns, initial imperfections were considered, applied by mean of horizontal forces in the y direction. The value of these forces was taken as equal to N/1 000, where N represents the axial force of the columns.

g) Spalling of concrete is unlikely to occur as the XC1 class is considered and the concrete strength is below 55 MPa in accordance with Reference [4].

h) During the cooling phase, the worst assumption was taken which is an additional strength loss of 10 %, complementary to the loss obtained at the maximum temperature seen by the concrete (consistent with Reference [19], Annex D).

i) Only the first floor was modelled in accordance with Reference [4], Section 2.4.3 which allows analysing a part of the structure with the condition that proper support and boundary conditions are chosen. For the sake of simplicity, if the 3D structure is replaced by a 2D frame, there are three degrees of freedom (DoF) to consider at the top of the heated columns where the limits to the model are set. Therefore, the following discussion can be done for every DOF.

   1) Vertical displacement (DoF): it was free in the model and there would be no difference if there were columns of the level above the fire compartment.

   2) Horizontal displacement: the restraint to horizontal displacement at the end of the slab can be evaluated by comparing the elastic stiffness to this displacement brought in the model, (EA/L of the slab + 12 EI/H^3 in the three heated columns) to the same stiffness if the columns above the fire compartment were included (same expression + 12 EI/H^3 in the three unheated columns). In this frame, it gives a ratio of 2,94/3,03 = 97 %. This is the amount of the "total" stiffness that was considered in the model. Nonetheless, the stiffness of the heated parts will decrease during the fire but, on the other hand, the stiffness of the columns above the fire compartment was calculated on the hypothesis that there is no rotation on the upper end of this column which is an overestimation.

   3) Rotation: it was the most affected DoF by modelling only first floor. It is considered that neglecting this additional stiffness is on the safe side because any additional stiffness in rotation introduced in the model would delay the failure by bending in the beams of the frame.

5.6.3 Structural behaviour of the building

5.6.3.1 General

This subclause presents the behaviour of the building when all elements were fully exposed to a fully developed fire with a decay phase. The edge beams and the exterior columns were partially exposed to fire.

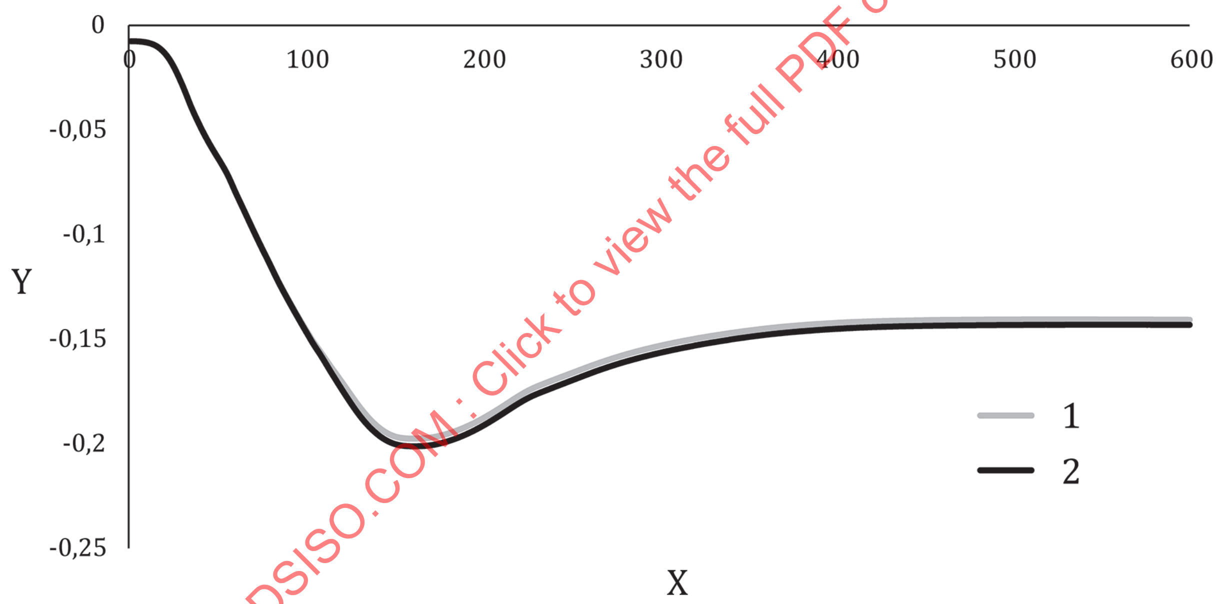
5.6.3.2 Displacements of the slab in fire conditions

This subclause demonstrates the vertical displacements of the slab exposed to the fire with a decay phase.

The vertical displacement increases constantly until approximately 160 min, reaching the maximum value of 0,20 m at Point 1-2/B-C. Even if the structure is symmetric after 100 min of fire exposure, some differences can be observed.

The maximum displacement is reached in all bays around 160 min.

For the first bay slabs (between axis 1-2/A-B and 1-2/B-C), the maximum displacement of 0,20 m is reached in the decay phase, once the temperature in the compartment starts to decrease. After reaching this value, the displacements decreases, reaching end value equal to 0,14 m after 400 min. No collapse was observed (see Figure 18).



Key

X time, in min

Y vertical deflexion of slab, in m

1 point 1-2/A-B

2 point 1-2/B-C

Figure 18 — First bay slab displacements

For the second bay slabs (between axis 2-3/A-B and 2-3/B-C), the maximum displacement is 0,196 m. During the cooling phase, the displacements recover to a value equal to 0,16 m.

For the third bay slabs (between axis 3s/A-B and 3s/B-C), the maximum displacement is equal to 0,193 m and recover to a value of 0,159 m in decay phase.

The distribution of the vertical deflections of the slab and the deflected shape of the building at different times are presented in Table B.2 and Table B.3.

5.6.3.3 Beam vertical displacements

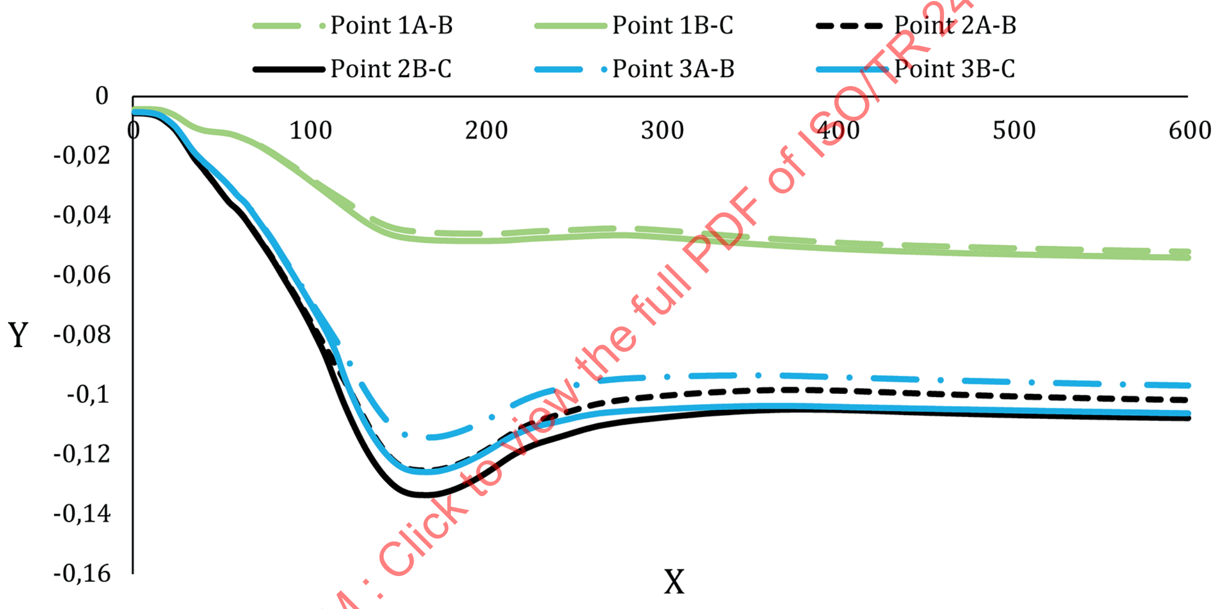
In this subclause, the displacements of the beam elements are presented.

Figure 19 shows that the maximum displacement is equal to 0,134 m for beam axis 2 in the span between B-C.

For the exterior beam (axis 1), the displacement increases reaching a value around 0,04 m in the first 140 min. This value is nearly constant until the end of analysis, with a slightly decreasing slope after 280 min, coming to the final displacement of 0,052 m displacement.

For the interior beams (axes 2 and 3), after reaching the maximum value of displacement around 160 min, a decrease in displacement can be noticed until 300 min of fire exposure, after which the displacement is nearly constant.

After 2 h of fire exposure, the displacements of the beams for the same span are different even though if the building is symmetric.



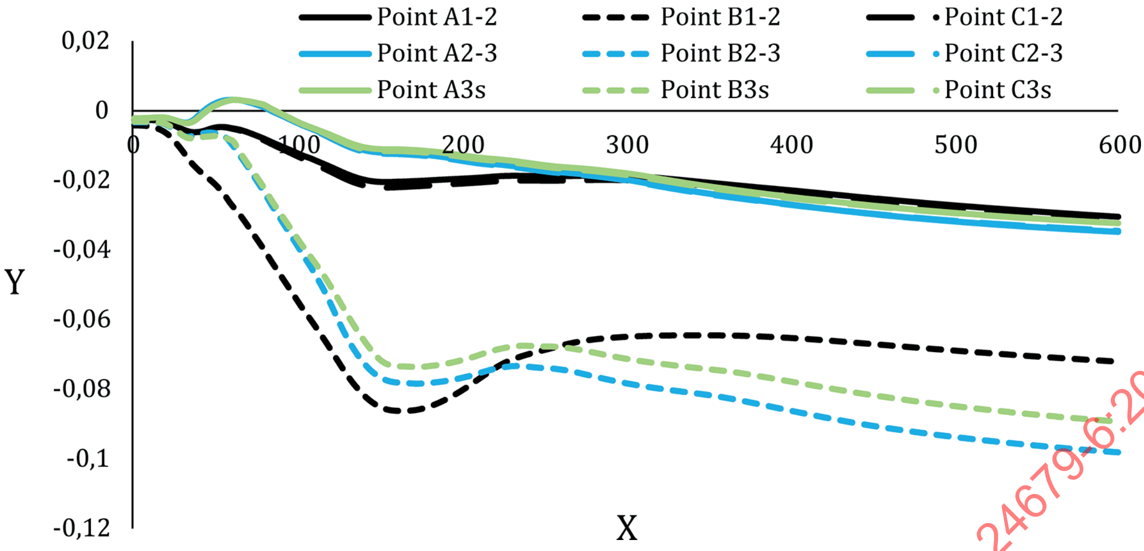
Key

X time, in min

Y beam displacement, in m

Figure 19 — Vertical beam displacements — Axes 1, 2, 3

Figure 20 shows the beam displacements in axes A, B and C. The interior beam axis B, span 1-2 experiences a maximum displacement of 0,086 after 163 min of fire exposure. After reaching this value, the displacement decreases until the value is almost constant during the analysis at one point. For the beam axisB_span2-3 and beam axisB_span3s, the displacement keeps increasing, except when at around 200 min, the displacement decreases for several minutes. For the beam in axes A and C, displacements are around 0,03 m after 10 h since the fire started.



Key

X time, in min

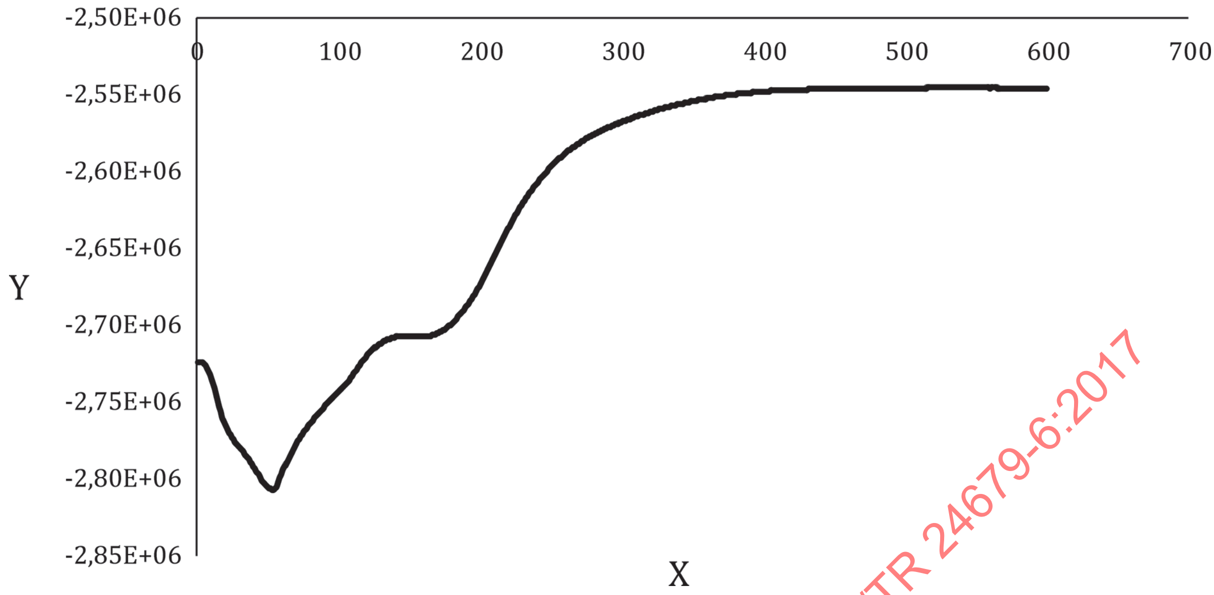
Y beam displacement, in m

Figure 20 — Vertical beam displacements — Axes A, B, C

5.6.3.4 Behaviour of the central column B2

5.6.3.4.1 Evolution of the axial force

Figure 21 shows the evolution of the axial force. At the beginning of the fire (1 min), the axial force is equal to 2 724 kN. After 1 h of fire exposure, the axial force increases to 2 797 kN. This increase is due to the fact that free expansion (which would be higher in this column heated on four sides than in other columns) is restrained by the stiffness of the surrounding beam elements. After 120 min of fire exposure, the axial force decreases (the concrete strength submitted to fire is reduced) and at the end of the analysis, the column is resisting a force equal to 2 546 kN.



Key

X time, in min

Y axial force, in N

Figure 21 — Evolution of the axial force of the central column B2

5.6.3.4.2 Column stresses for the top part of central column

Table 6 presents the stresses for the top part of the central column B2.

At the beginning of the analysis, 1 min after fire exposure all the section is in compression.

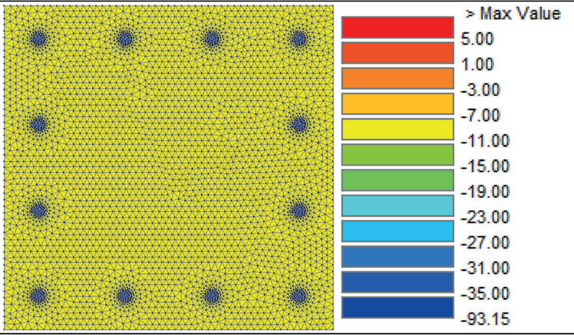
After 60 min of fire exposure, the exterior part is extending because it is heated. However, the concrete in the interior is cold so a tensile load is applied by the extended part. At this time, the compressive stress is therefore increasing in the exterior part and decreasing in the middle, eventually becoming tensile stresses after 2 h.

At 160 min of fire exposure, the thermal gradient is moving inside the section a large section of the middle part is heated and the tension zone area is reduced compared to the previous case. At the same time, the fire in the compartment is in its decay phase so the exterior part of the column is starting to cool down, this means that tension forces will appear.

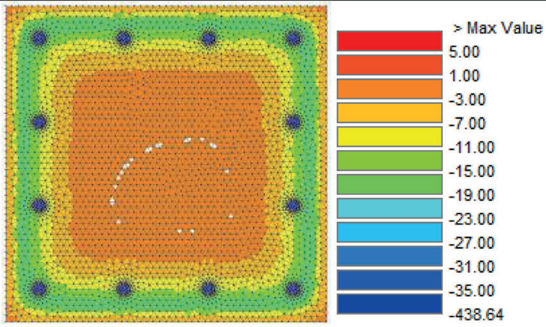
For 400 min and 600 min of fire exposure, excluding the exterior part of the section which is cracked, the remaining fibres are in compression, and the maximum compression force is in the central area of the column.

The reason that corner rebars are in tension is due to the fact that the internal part of column is hotter than external parts and then it has to equilibrate the dilatation, so a tensile load is impose by the internal part.

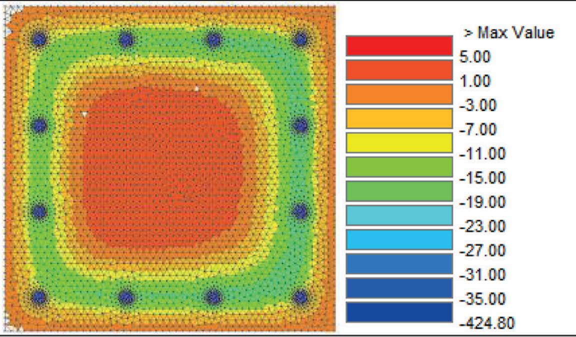
Table 6 — Stress distribution for the top of the central column B2



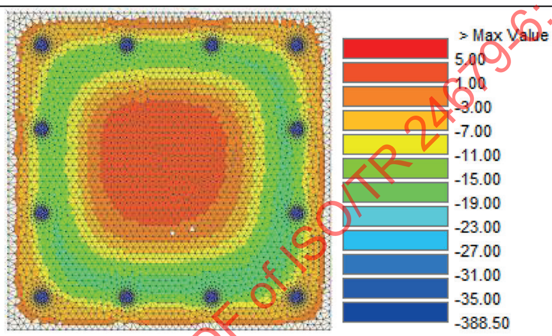
1 min



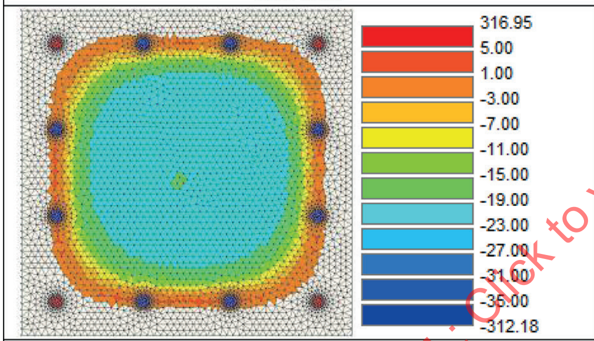
60 min



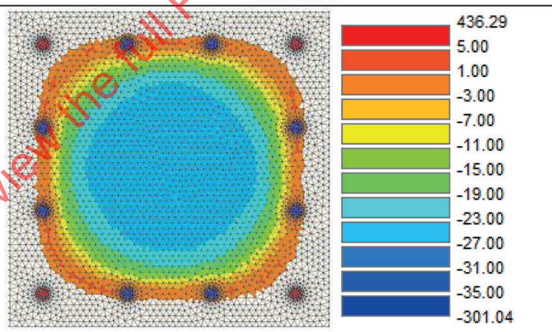
120 min



160 min

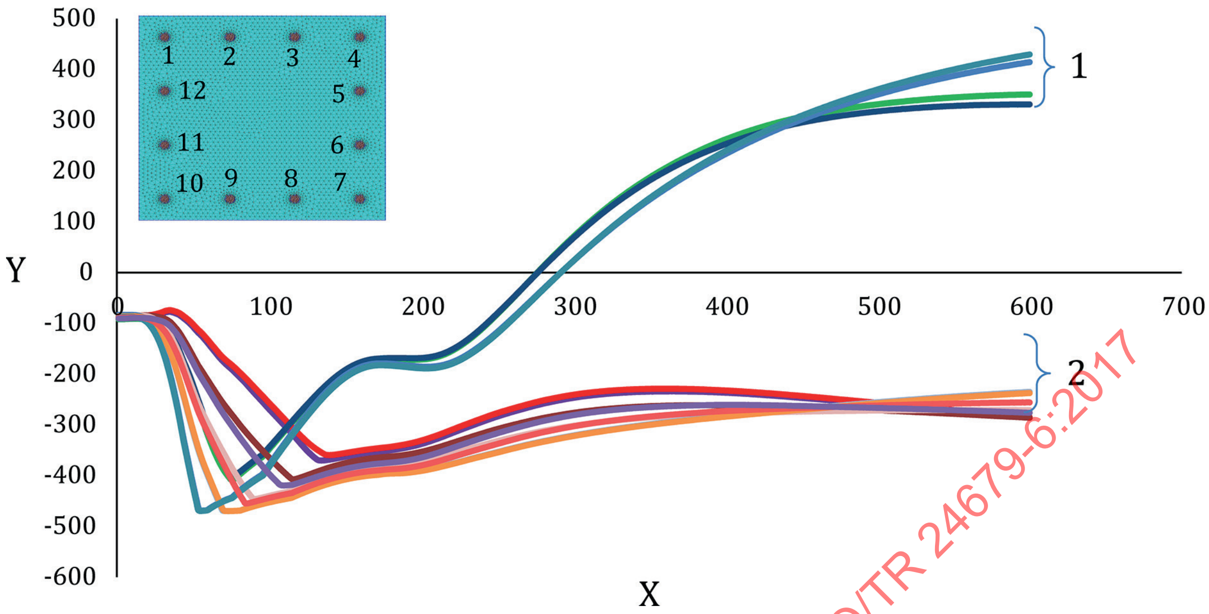


400 min



600 min

Figure 22 shows the evolution of stresses for corner reinforcements (rebar 1, rebar 4, rebar 7, and rebar 10) and for middle rebars (rebar 2, rebar 3, rebar 5, rebar 6, rebar 8, rebar 9, rebar 11, and rebar 12). It is apparent that all the rebars work in compression, with the exception of the corner rebars, when they are in tension are around 300 min.



Key

X time, in min

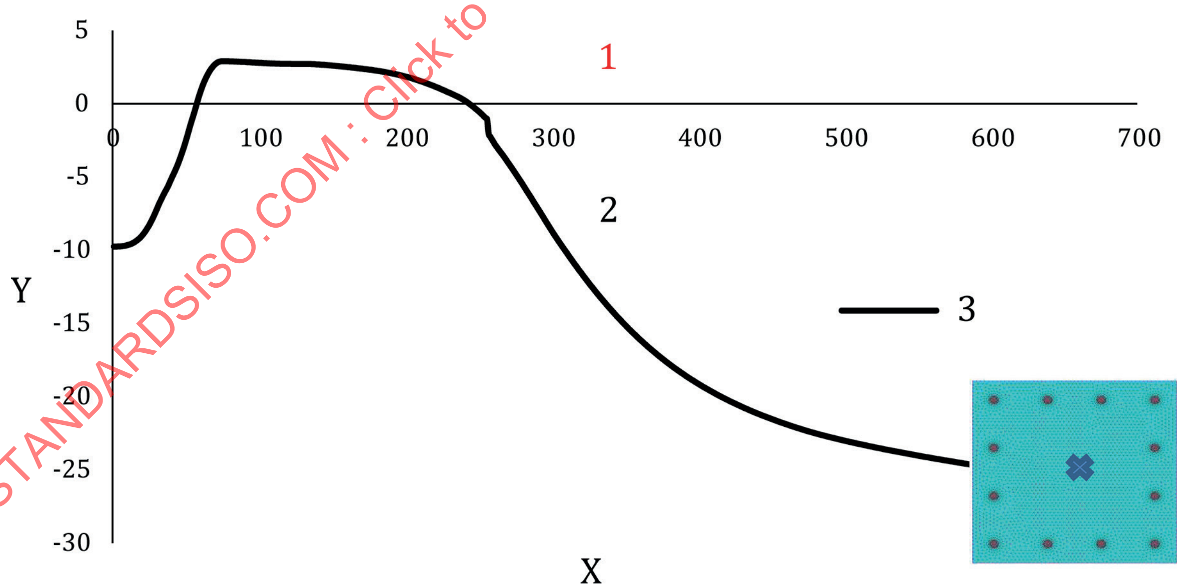
Y stress, in N/mm^2

1 corner

2 middle

Figure 22 — Rebar stresses — Central column (top)

Figure 23 shows the evolution of stresses in the centre of heated section.



Key

X time, in min

Y stress, in N/mm^2

1 tension

2 compression

3 centre of the column

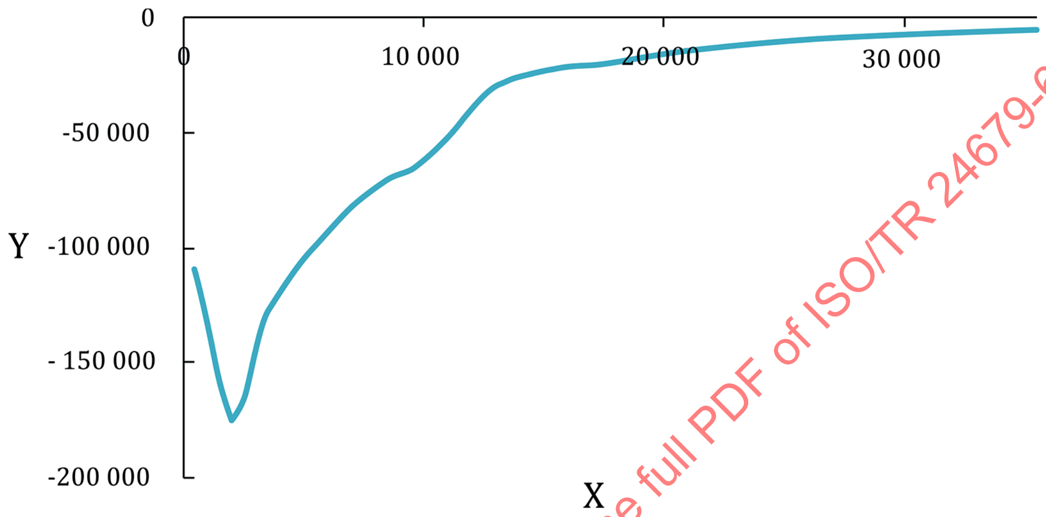
Figure 23 — Evolution of the concrete stresses in the centre of the section

5.6.3.5 Shear capacity

The shear capacity was calculated in order to compare to the fire-induced shear forces.

The first step consists in determining the maximum fire induced shear forces in the overall structure during the entire fire duration. Results of detailed structural analysis show that for columns, the maximum shear force is registered for corner columns A1 and C1 (local axis Z) with a maximum of 210 kN and for column B1 (local axis Y) with a maximum of 188 kN.

Figure 24 shows the fire induced shear forces in the beam located in axis 2 with a maximum value of 176 kN after 1 985 s (i.e. 33 min) of fire exposure.



Key

X time, in s

Y shear, in N

Figure 24 — Fire induced shear forces (Vy) for the beam located in axis 2

Given the shear reinforcement ratio which is (Ø 12, 200 mm) for columns and (Ø 8, 175 mm) for beams, it can be concluded that the most critical situation occurs for the beam in axis 2, for which further analysis was performed.

However, it should be noted that the maximal shear force is not necessarily concomitant with the maximal temperature in the stirrups. As the temperature along the stirrup is not uniform at a given time, the assumption of the time where the temperature is the maximum in the bottom rebars in the beam (around 10 000 s or 165 min) was taken. Once the calculation was performed, it appears that this case was not critical compared to the case where the shear force was maximum at 33 min. The later point is presented hereafter.

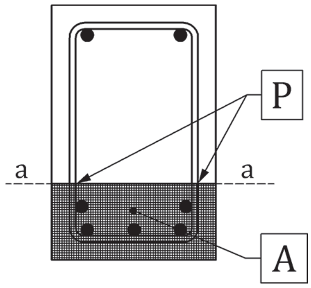
The following steps were thus followed in accordance with Reference [4], Annex D:

— the reduced geometry is equivalent to the initial geometry, as the temperature is below 500 °C in the whole section at 33 min (i.e. b_{fi} = 25 cm and h_{fi} = 40 cm);

— the reference temperature, θ_P, in stirrups as the temperature in the point P (intersection of Section a-a with the stirrup, see Figure 25) is equal to 20 °C (the height of the effective tension area is equal to 10,5 cm);

— the reduction of design strength of steel in links should be taken with respect to the reference temperature

$f_{sd,fi} = k_s(\theta)\, f_{sd}$ (20)



Key

A effective tension area

Figure 25 — The reference temperature for the stirrup should be evaluated at points P[6]

Then, the calculation methods for design and assessment of shear, as in Reference [10] were applied directly to the reduced cross-section, as given in Formula (16):

$$V_{\mathrm{Rd,fi}} = \min\{V_{\mathrm{Rd,sfi}} = (A_{\mathrm{sw}}/s)\cdot z_{\mathrm{fi}}\cdot f_{\mathrm{ywd,fi}}\cdot\cot\theta;\ V_{\mathrm{Rd,max}} = \alpha_{\mathrm{cw}}\, b_{\mathrm{w,fi}}\, z_{fi}\, v_1\, f_{\mathrm{cd,fi}}/(\cot\theta + \tan\theta)\} \quad (16)$$

where

A_{sw} is the cross-sectional area of the shear reinforcement;

s is the spacing of the stirrups;

$f_{\mathrm{ywd,fi}}$ is the design yield strength of the shear reinforcement;

θ is the angle between concrete compression struts and the main tension chord, in the chapter concerning Limit State Design (ULS-SLS) of the workshop “Design of Concrete Buildings”[11], it has been chosen $\cot\theta = 2{,}5$;

v_1 = $0{,}6(1\text{-}f_{\mathrm{ck}}/250)$;

$\alpha_{\mathrm{cw\ i}}$ is a coefficient to take into account the stress state and equals 1 for non-prestressed structures;

$V_{\mathrm{Rd,sfi}} = (2 \times \pi \times 4^2/175) \times 330 \times 500 \times \cot 21{,}8 = 237$ kN

$V_{\mathrm{Rd,max}} = 1 \times 250 \times 330 \times 0{,}6\ (1\text{-}25/250) \times 25/(\cot 21{,}8 + \tan 21{,}8) = 384$ kN

$\rightarrow V_{\mathrm{Rd,fi}} = 237$ kN $> V_{\mathrm{Ed,fi}} = 176$ kN

The calculation shows that the shear capacity is sufficient to withstand the fire induced shear force.

5.6.3.6 Rotational capacity

The simplified procedure for continuous beams and continuous slabs is based on the rotation capacity of beam/slab zones over a length of approximately 1,2 times the depth of section. It is assumed that these zones undergo a plastic deformation (formation of yield hinges) under the relevant combination of actions. The verification of the plastic rotation in the ultimate limit state is considered to be fulfilled, if it is shown that under the relevant action the calculated rotation, θs, is less than or equal to the allowable plastic rotation under fire conditions[10]. However, no critical value for the allowable plastic rotation under fire condition is given in Reference [4].

As such, the suggested value given in the French national Annex of Reference [9] was selected. For steel class B, the allowable rotation under fire condition is given as 250 mrad.

The beam is 40 cm deep and the slab is 18 cm thick. The rotation should be checked over a length of 1,2 times the depth at 0,6 h from the support which means, respectively, at 24 cm and 11 cm from the support for the beam and the slab. Due to the mesh size, the check is performed at 45 cm from the support which definitely cover the previously mentioned distances.

The verification was performed for all the supports, some examples are pointed in Figure 26.

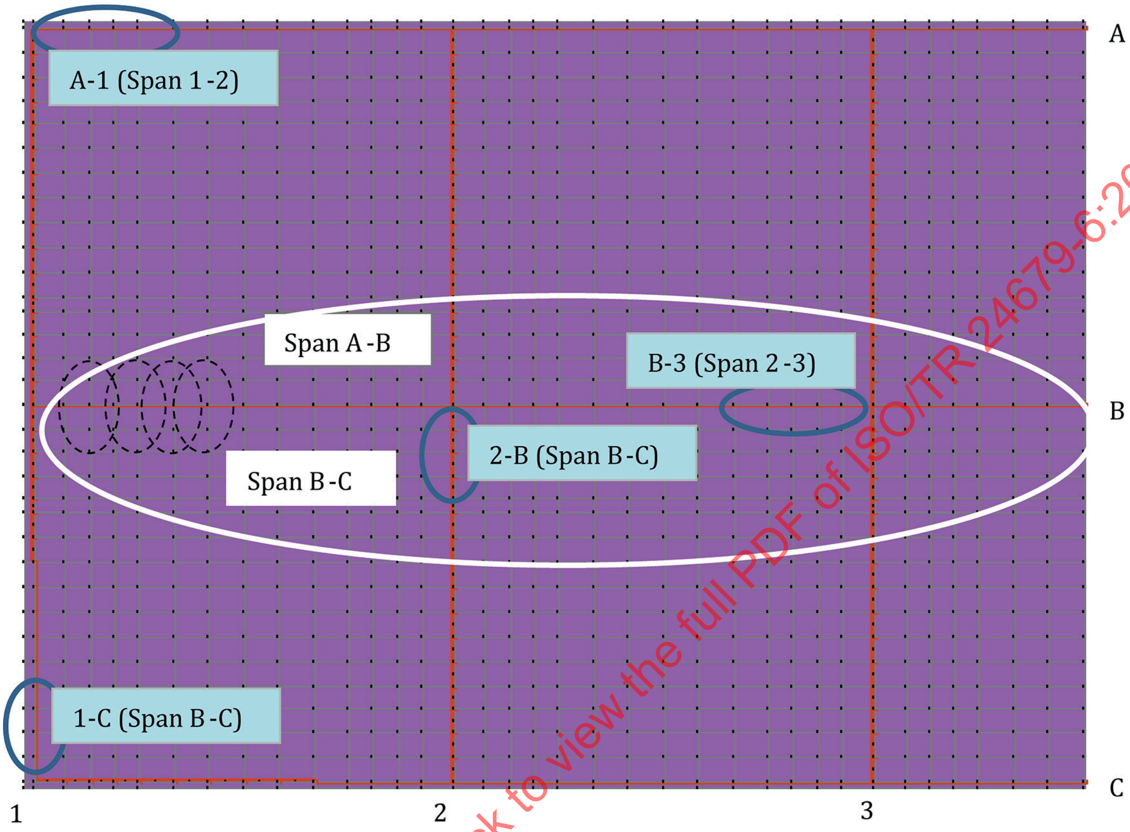
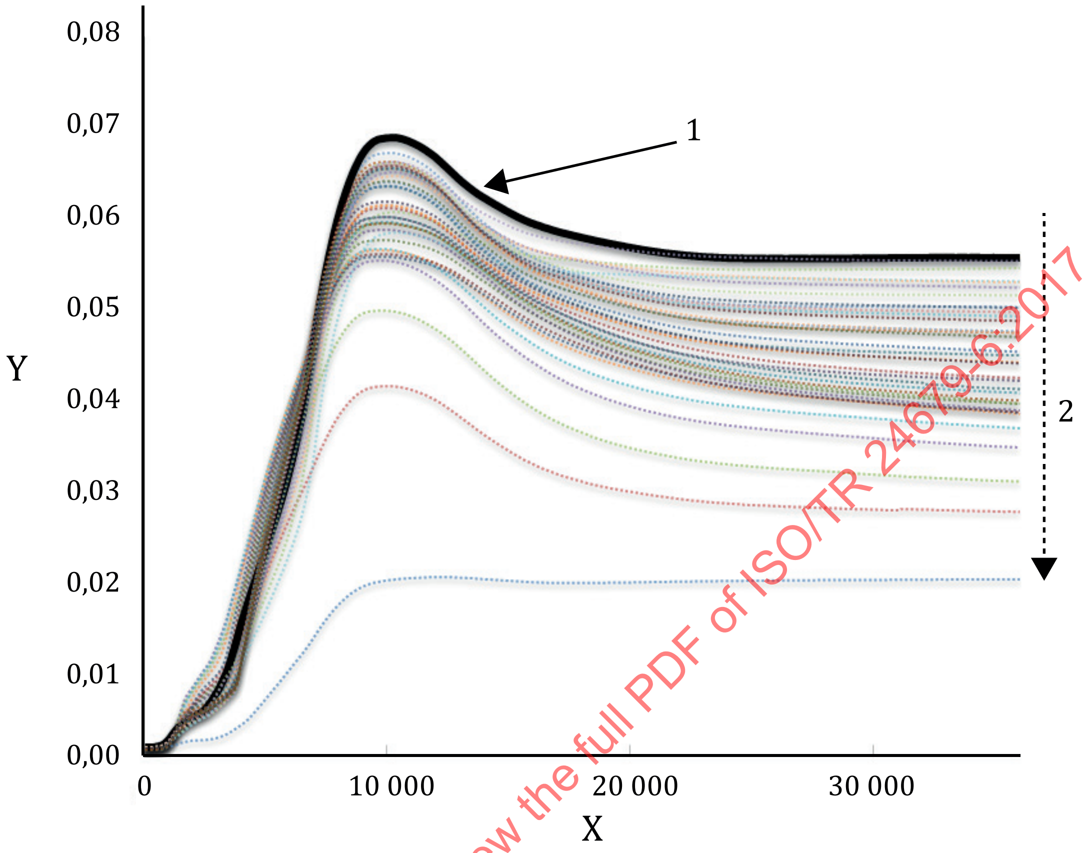


Figure 26 — Examples of supports whose rotational capacity has been checked (all the supports were checked)

Regarding the slabs, the maximum rotations appear for span B-C, along axis B. However, the maximum value is 70 mrad (see Figure 27) which was below the critical value (i.e. 250 mrad).



Key

X time, in s

Y rotation, in rad

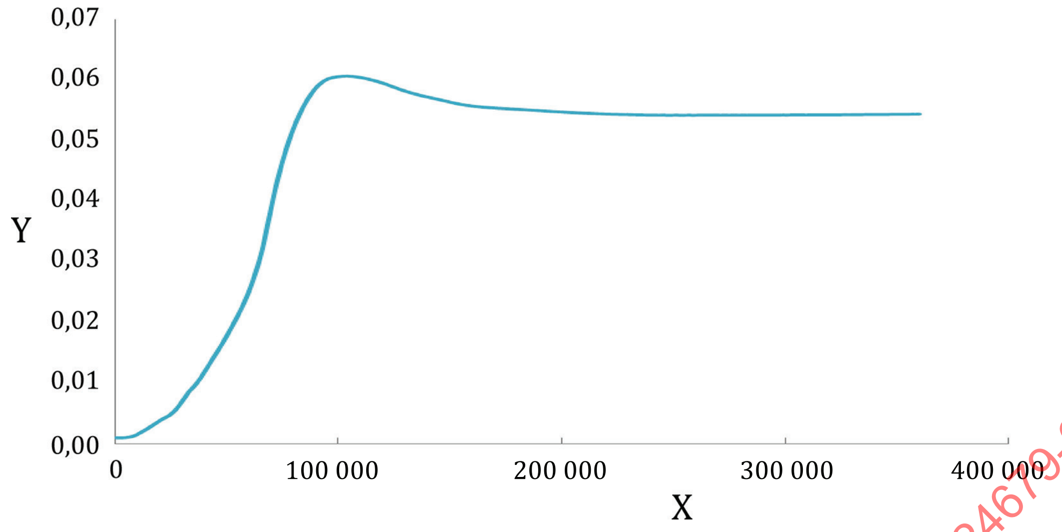
1 maximum rotation along axis B, side span B-C

2 rotation is decreasing near the supports axis 1, 2 and 3

NOTE No critical case is found.

Figure 27 — Rotations at 45 cm from the support beam axis B, span B-C, each curve corresponds to one mesh

Regarding the beams, the maximal rotations appear for axis 2, span B-C. The results show that the maximum value is 60 mrad (see Figure 28) which is below the critical value.



Key

Y time, in s

X rotation, in rad

Figure 28 — Rotation of the beam axis 2, span B-C

Results shows that no plastic hinges appear during the entire duration of the fire.

5.7 STEP 7: Assessment against the fire safety objectives

The performance of structure in case of fire was assess and it was shown that:

— no overall failure of the building, due to the loss of stability of columns, shear failure, rotational capacity was occurred; maximum deflection of all slabs does not exceed 1/20 of their spans;

— the shear capacity is sufficient to withstand the fire induced shear force and rotational capacity does not exceed 250 mrad.

Results of analysis then show the design fire objectives, for the relevant worst case design fire scenario are met with the available trial plan for fire safety of structure.

5.8 STEP 8: Documentation of the design for fire safety of structures

This document is prepared for the implementation of ISO 24679-1. As such, the steps outlined in ISO 24679-1 have been followed.

a) Interested and affected parties include the owner of the building, tenants of office area and the designer.

b) Scope of the project: an eight-storey concrete frame office building, see 5.1 for more details.

c) Objectives, functional requirements and performance criteria for the fire safety of structures were defined according to the type of occupancy, the properties of the structure as well as the requirements of national codes and standards. See 5.2 for the details.

d) Trial design plan for fire safety of structures: The office building studied here has been designed for the ambient temperature. Structural elements are not protected by any passive or active fire protection systems. For more details see 5.3.

e) Design fire scenarios and design fires: The relative impact of a fully developed fire was studied according to the history of the real fires in an office. A group of potential fire scenarios were examined and the worst ventilation condition and the acceptable fuel load density in this building were identified. The critical design fire was then produced. See 5.4 for more details.

f) Assessment methods: A group of potential fire scenario was compared and the critical design fire was produced using zone model. Detailed structural analysis then was carried out.

g) Data sources: Data were taken from ISO and/or corresponding national standards, codes, SPFE handbook, reference books in fire safety engineering and peer-reviewed literatures. For more details, see Bibliography.

h) Evaluation of the assessment results: are presented in thermal and mechanical analysis sections. Evaluation of the results shows the design fire objectives, for the worst case design fire scenario, are met with the available trial plan for fire safety of structure.

Summary and conclusions are as follows.

a) Analyses were performed in order to determine whether the design fire objectives, for the relevant design fire scenarios are met with the available trial plan for fire safety of structure.

b) In this respect, the plausible worst design fire scenarios were defined and the performance of structure in case of fire was assessed. Different performance criteria were checked and the results of this study show that this concrete building provides a sufficient level of safety in terms of structural behaviour in the event of fire.

5.9 STEP 9: Factors and influences to be considered in the quantification process

5.9.1 Material properties

5.9.1.1 Thermal properties

The thermal and physical properties for thermal transfer were taken into account as presented in 5.5. These values were considered as a function of temperature in the heat transfer calculation.

5.9.1.2 Mechanical properties

Mechanical behaviour calculations considered relevant data for mechanical properties as a function of temperature as presented in 5.6.

5.9.1.3 Uncertainty of material properties

The nominal properties quoted normally, and variations in thermal properties of structural materials have a degree of uncertainty as stated in ISO 24679-1. For example, moisture content and phase changes influence the variation in thermal properties of structural materials. The impact of uncertainties was studied through a sensitivity analysis study. The sensitivity analysis determined the relative impact of numerical, physical and building parameters on the performance of the structure. It was the intent of this sensitivity study to identify the important parameters that should be considered in design[1][3].

As such, a range of input variables in fully developed fire and heat transfer calculations were investigated. The parameter values for the base case scenario and the ranges investigated are given in Table 7. The base case values are used, unless specified otherwise.

A novel fire safety analysis for a large number of fire scenarios was carried out by a sensitivity analysis of the "base case" scenario using an OAT method in order to address the uncertainty of material properties and to identify a number of scenarios which results in a temperature greater than the critical rebar temperature.

Table 7 — Parameter values for the base case and range of parameters investigated in the heat transfer model

Parameters	Unit	Base case	Range	Reference	Comment
Axis distance for reinforcement (d_r)	mm	44	[40 to 48]	[11]	Range taken to be representative of the typical range found in real buildings. Base case value as per the design of the building.
Density (ρ)	kg/m^3	2300	[1 900 to 2 300]	[4],[13], [20]	Range taken to represent concrete densities ranging from light weight concrete to normal concrete. Base case value is taken as siliceous concrete density from Reference [4].
Thermal Conductivity (K)	W/m·K	1,33	[1 to 1,95]	[4],[6],[13]	Range taken for representative values of light and normal weight concrete thermal conductivities. The base case value is taken as the lower limit curve of thermal conductivity of normal weight concrete in ambient temperature from Reference [4].
Specific heat (C)	J/kg·K	900	[840 to 1 100]	[4],[6],[13], [20]	Range taken to represent limits of concrete specific heats. The base case value is taken from Reference [4].
Convective Coefficient – Exposed Surface (h_c)	W/m^2·K	35	[10 to 50]	[5],[13]	Range taken to represent limits in a fire condition. Base case value is taken from Reference [5].
Emissivity (ε)		0,7	0,5 to 1	[4],[5],[13]	Range taken to test sensitivity; however values in an accidental fire are expected to be above 0,5. Base case value is taken from Reference [5].

In the OAT method, only one input variable in the base case fire scenario is changed, keeping others at their base case value in order to observe how changing one input variable affected the output, particularly the rebar maximum temperature (RMT) and time to reach the rebar maximum temperature (t_{RMT}). In OAT sensitivity analysis, the input parameters were swept across the range investigated.

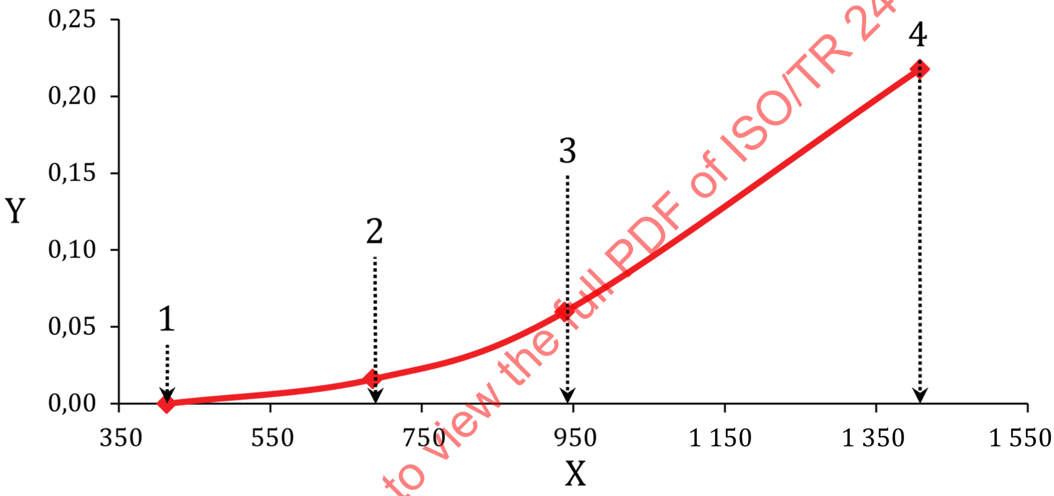
The results from the OAT sensitivity analysis for the parameters in Table 6 show the variation of the maximum rebar temperature with changes in the input parameters (Figure C.1 to Figure C.6). The results demonstrate that the peak rebar temperature and the corresponding time are slightly or moderately affected by the variation of input parameters. It should be noted that no critical case (i.e. design fire) was found in the range of the parameters. A concrete cover of 44 mm[12] is used for the base case; in addition, the range selected implicitly included the possible loss of 10 % concrete cover due to spalling as illustrated in Figure C.1.

Complementary to the previous steps, the design fires due to the stochastic variation of input variables in Table 6 was evaluated by the assessment of the failure probability of the selected concrete slab[1]. These analyses were achieved using Monte Carlo simulation (variation of all input variables). The Monte Carlo simulation is a method that performs multiple experiments using a large number of randomly generated samples from the input space, and belonging to a certain probability distribution.

For the purpose of the Monte Carlo simulation, a Gumbel distribution was assumed for the fire load density in accordance with Reference [7] and a uniform distribution was assumed for the other input parameters for the selected ranges. A random value was selected for each of the input variables based on the range and the maximum rebar temperature in the concrete beam was calculated and results were recorded and process were repeated. The probability of failure was calculated by $p_f = n_f/n$, where n_f is the number of failed simulations and n is the total number of simulations.

Figure 29 shows the probability of reaching the failure criteria in a concrete beam 250 mm × 400 mm for different design fire load densities due to the uncertainty of material properties.

The variation of fire load densities in x-axis could be linked to the impact of the availability or unavailability of active fire protection setups in accordance with Reference [5]. The possible event sequence arising from availability or lack of active fire protections were structured in Figure 30. Each final event is an aggregation of events and was assigned a fire fighting measures index in accordance with the Reference [5]. The indexes were multiplied to the base case design fire load density and were linked to the values in Figure 29.



Key

X design fire load, in MJ/m²

Y probability of failure

1 failure of detection system

2 failure of sprinkler or detection

3 failure of sprinkler and detection system

4 failure of sprinkler, detection system, safe access route, and no fire fighting intervention

Figure 29 — Probability of failure for a range of active fire fighting measures from a Monte Carlo simulation based on 1 500 trials

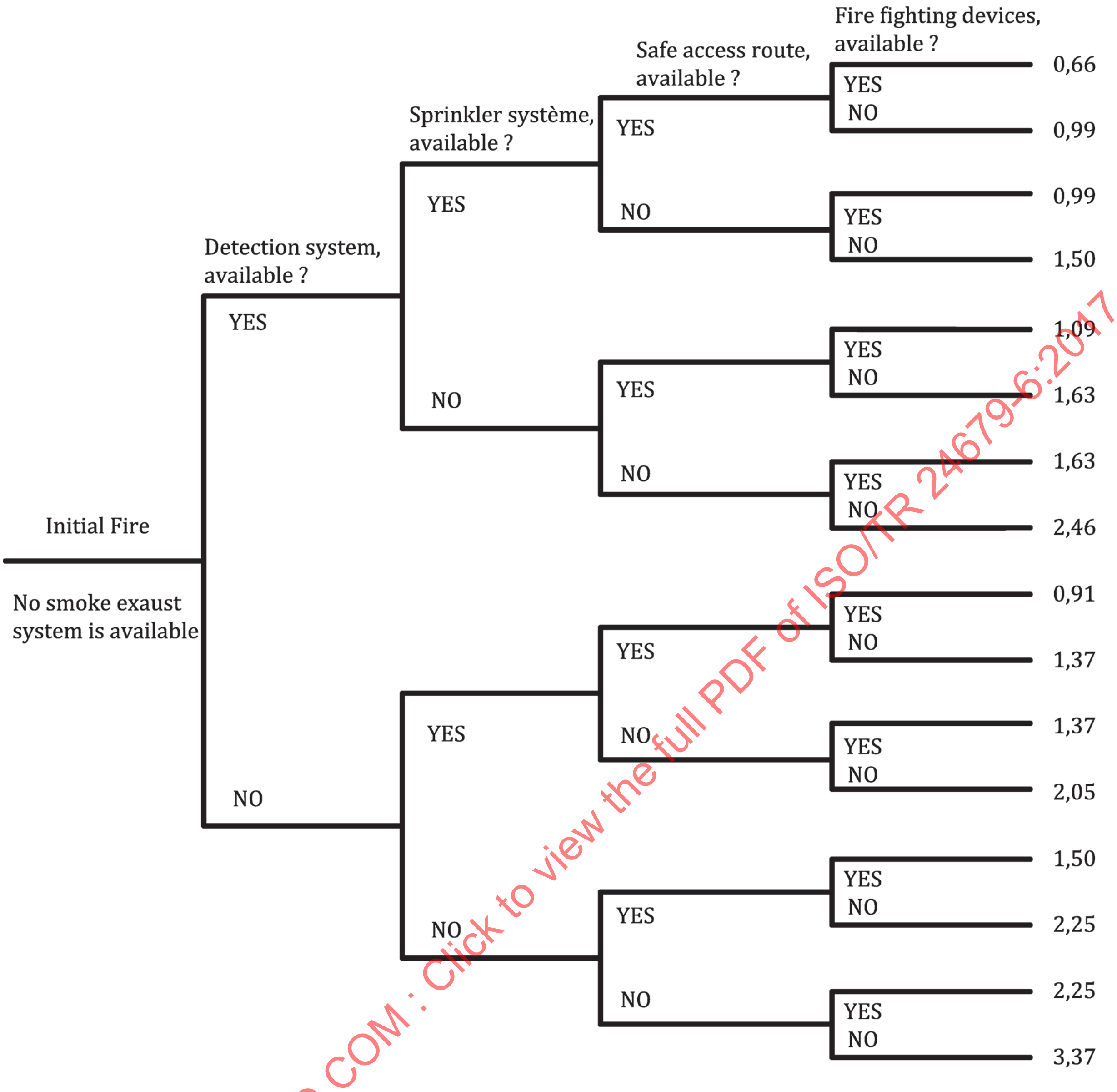


Figure 30 — Event tree and possible scenarios for Monte Carlo sensitivity analysis

The study demonstrates that the presented methodology can provide a comprehensive understanding of the factors affecting the structural fire resistance and inform fire development and detailed structural analysis.

It should be noted that in order to fully understand the influence of certain (stochastic and/or deliberate design changes in) parameters, a full probabilistic analysis could be carried out.