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Fire safety engineering — Performance of structures in fire —

Part 8: Example of a probabilistic assessment of a concrete building

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

Partie 8: Exemple d'évaluation probabiliste d'un bâtiment en béton

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Contents

Page

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*.

A list of all parts in the ISO 24679 series can be found on the ISO website.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

This document provides 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 clauses of ISO 24679-1 which are considered relevant to this document are identified and the clause titles are kept the same and in the same order.

The purpose of this document is to demonstrate the application of the steps outlined in ISO 24679-1 for fire safety engineering, performance of structures in fire, applying probabilistic methods.

The analysis shows how the achievement of the fire safety objectives, with respect to structural fire resistance, can be demonstrated through probabilistic analysis. The building is based on a demonstration case for Eurocode 2[2] and is thus conformant with the design requirements of EN 1992-1-2[5]. For this type of building, a probabilistic analysis would generally not be performed. However, probabilistic analysis can demonstrate the achievement of the fire safety objectives for situations which are not conformant with standard design guidance.

This document only presents an example application of a probabilistic analysis. More advanced applications considering system behaviour and stochastic fire exposure are possible. These more advanced procedures will generally result in an improved understanding of the reasonably foreseeable structural behaviour in case of fire, and can, for example, be used for an in-depth analysis of the post-fire structural performance.

Probabilistic methods make engineering assumptions more explicit. This pushes the engineer to question their competence and promotes an in-depth communication with stakeholders on the intended structural performance in case of fire.

Fire safety engineering — Performance of structures in fire —

Part 8: Example of a probabilistic assessment of a concrete building

1 Scope

This document provides an example of a probabilistic assessment of a concrete building by revisiting the structural fire analysis of the concrete building presented in ISO/TR 24679-6, using probabilistic approaches. Specifically, the most heavily-loaded concrete column is analysed probabilistically, using the evaluation in ISO/TR 24679-6 as a starting point.

This report only addresses the fire safety objectives related to the structural performance. The analysis within this document therefore forms 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, *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 terminology databases for use in standardization at the following addresses:

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

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

3.2 Symbols

e average eccentricity

E load effect

E_d design value of E

E_k characteristic load

f out-of-straightness

f_{ck} characteristic concrete compressive strength

f_{yk}	characteristic steel yield strength
G	dead load
G_F	dead load façade
G_k	characteristic value of the permanent load effect
K_E	model uncertainty for the load effect
K_R	model uncertainty for the resistance effect
L	column height
$N_{ED,fi}$	design load
p_s	reliability (i.e. probability of failure in the case of a given fire exposure)
p_f	failure probability
$p_{f,t}$	target maximum failure probability
P	axial load
P_{max}	load bearing capacity of the column
$P_{max,num}$	numerical evaluation of P_{max}
P_{Gk}	characteristic permanent load
P_{Qk}	characteristic imposed load
P_T	total axial load
Q	dominant live load effect
Q_k	characteristic value of the imposed load effect
R	resistance effect
R_d	design value of R
R_k	characteristic resistance
V	coefficient of variation
V_E	coefficient of variation for the load effect
V_R	coefficient of variation for the resistance effect
Z	limit state function
β	reliability index
β_t	target reliability index
γ_0	safety factor
γ_E	load factor
γ_R	resistance factor

ε	surface emissivity of the member
μ	mean value
μ_E	mean value for the load effect
μ_R	mean value for the resistance effect
σ	standard deviation
σ_E	standard deviation for the load effect
σ_R	standard deviation for the resistance effect
Φ	out-of-plumbness; the standard normal cumulative distribution function
χ	load ratio (characteristic live load effect relative to total characteristic load effect)
Ψ	combination factor for the live load effect
Ψ_{fi}	fire design variable action combination factor
Ø	reinforcement bar diameter

4 Design strategy for fire safety of structure

The built environment of this example is an office building, as considered in ISO/TR 24679-6. The structural elements are composed of concrete.

For the concrete columns, the tabulated fire resistance, under standard thermal action (ISO 834) in accordance with EN 1992-1-2 is 90 min, while the calculated fire resistance using simplified calculation methods is 180 min, as specified in Eurocode 2.[2]

The safety level (i.e. probability of failure) associated with a tabulated or calculated standard fire resistance is not known. Consequently, there is a possibility that the structure does not behave as expected during fire exposure, notably because:

— the expectations did not account for the failure probability;

— the real fire conditions and structural behaviour do not match the concept of fire resistance under standard fire exposure.

These shortcomings can be reduced by:

— conducting a detailed analysis taking into account potential fire scenarios and structural behaviour for the building system in question, as applied in ISO/TR 24679-6, where the fire was defined taking into account Reference [3]; or

— conducting a probabilistic assessment of the failure probability for an isolated structural element exposed to a standard fire, as applied further; or

— a combination of both of the previous bullet points, for example, a full probabilistic analysis of a structural system, taking into account uncertainties in the fire development and structural response. This level of analysis can be very computationally expensive.

In the following clauses, a probabilistic assessment is carried out for the example concrete building (specifically, for the most loaded concrete column), demonstrating confidence in the achievement of the fire safety objectives.

5 Quantification of the performance of structures in fire

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

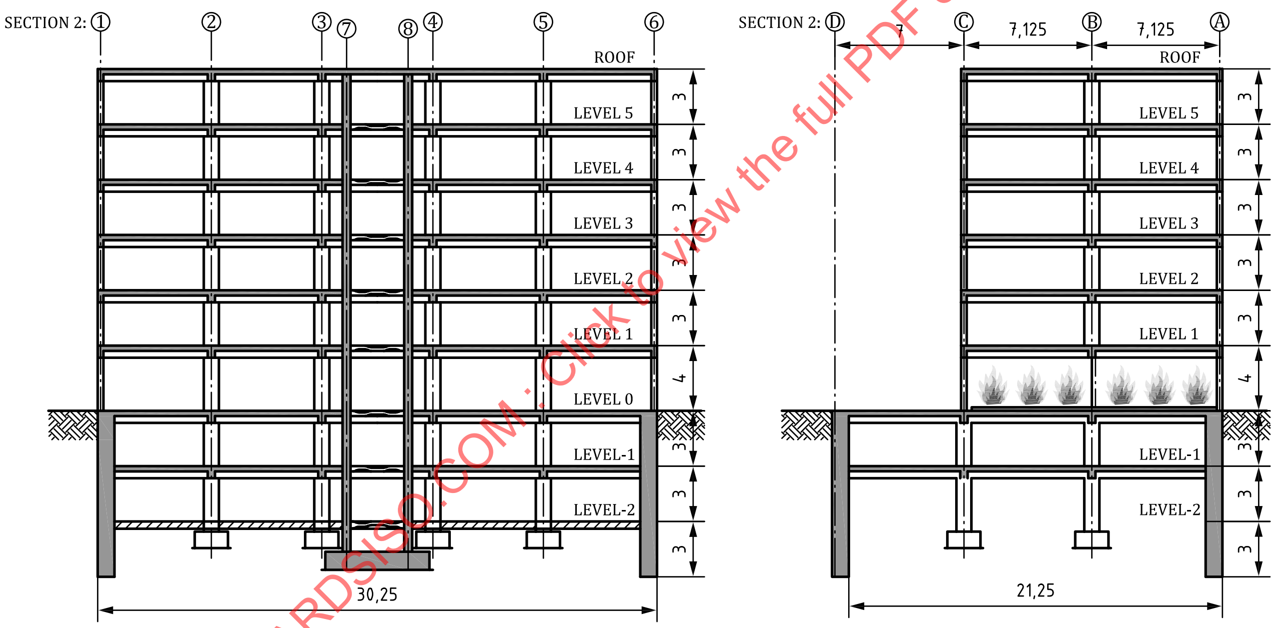
5.1.1 Built-environment characteristics

The concrete building considered is the same as that studied in ISO/TR 24679-6. The building characteristics are re-introduced in this subclause.

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 × 14,25 m wide × 25 m high. The ground floor has a height of 4 m, whereas the upper storeys have a height of 3 m. Elevators and stair cases are placed in the central core.

The length is divided into five structural bays and the width into two bays. Each bay measures 6 m × 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 × 500 mm wide, and the beams are 400 mm deep × 250 mm wide.

Dimensions in metres



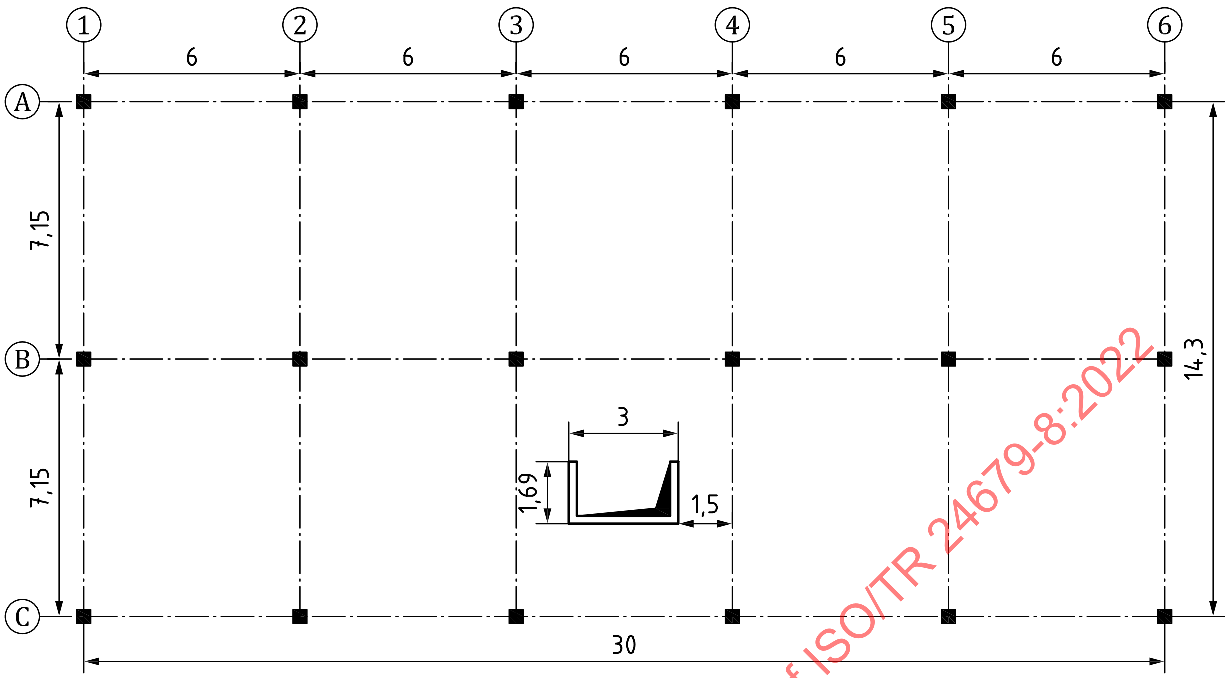


Figure 1 — Plan and elevation of the structure

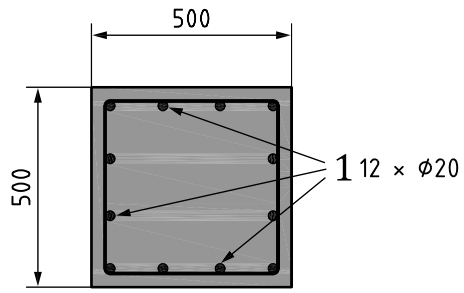
The structure includes three kinds of structural members: reinforced concrete columns, beams and slabs. The cross-section of the column is equal to 0,25 m^2 and is presented in Figure 2 (Key element 1, longitudinal reinforcement).

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.

Dimensions in millimetres



Key

1 longitudinal reinforcement

Figure 2 — Column cross-section

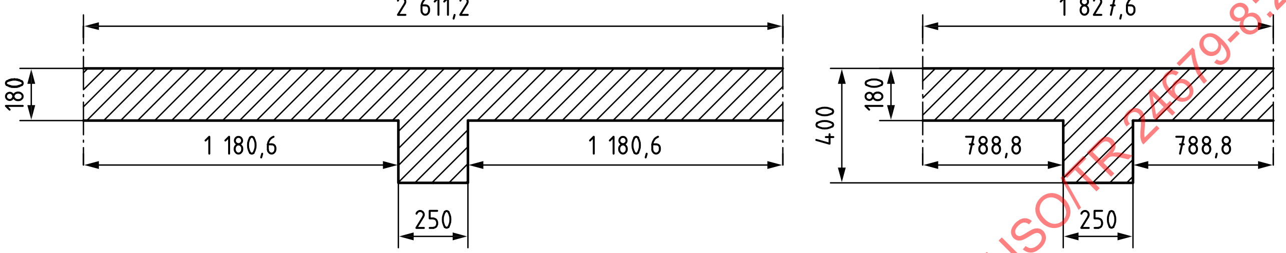
The reinforcement in the column and the axis distance are presented in Table 1.

Table 1 — Column reinforcements and the axis distance of reinforcements.

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

In Figure 3, the cross-sections of the beams are illustrated, and in Figure 4 the concrete slab and reinforcement is presented (180 mm thick). As the analysis further focuses on the most-loaded concrete column, no further details regarding the beams and slabs are given here.

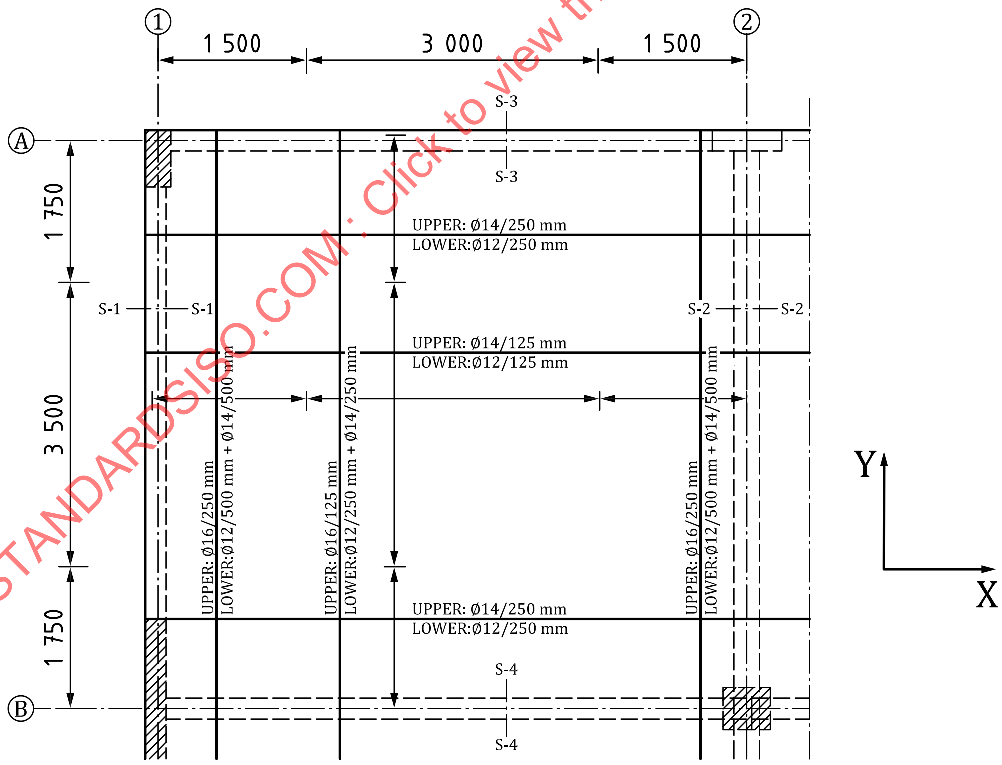
Dimensions in millimetres



a) Cross-section at mid-span **b) Cross-section at intermediate support**

Figure 3 — Continuous beam cross-section

Dimensions in millimetres



NOTE The terms "UPPER" and "LOWER" in this figure refer to the upper side and lower side measurements, respectively.

Figure 4 — Reinforcement distribution in the slab

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.

Commonly, structural fire performance requirements are stated with respect to the ISO 834 standard fire curve. This is also the approach followed in the analysis below. For a consideration of fire development through parametric (natural) fire exposure, reference is made to ISO/TR 24679-6.

The application of the ISO 834 standard fire curve has the advantage of clarity and ease of communication, as no further engineering assumptions nor calculations have to be made. This comes at the cost of reduced correspondence with actual compartment fire development. Notably, the ISO 834 standard fire ought to be considered as a reference exposure, not as a realistic representation of fire.

In other words, performance relative to the ISO 834 standard fire exposure is applied as a well-understood proxy for performance during any of the wide range of natural fires. Capturing a severe fire exposure, the standard fire exposure is set at 240 min (4 h) of ISO 834.

5.1.3 Mechanical actions

Dead and live loads are presented in Table 2.

Table 2 — 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 concrete column with the highest design load is considered further. The design load is determined in a single (deterministic) analysis considering the load combination using Formula (1), in accordance with Reference [4]:

$$G+\psi_{2,}Q \tag{1}$$

where

G is the permanent load effect;

Q is dominant live load effect;

$\psi_2 = 0{,}6$ is the combination factor, accounting for the low likelihood of fire coincidence with a high realization of the live load effect.

The distribution of loads between the columns was calculated using an advanced model (finite element model in 3D, see ISO/TR 24679-6). The loads applied in this model are summarized in Table 3.

Table 3 — Used loads in detailed structural analysis

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

Table 3 *(continued)*

				Unit
Interior beams	Dead load (G)	0,25 × 0,40 × 25	2,5	kN/m
	Total load	$\boldsymbol{G}$	**2,5**	

The resulting loads on the columns are indicated in Figure 5. The most loaded concrete column is evaluated further as a critical element, taking into account pinned connections for further analysis in isolation. The evaluation of the critical column as pinned ignores redistribution to other structural members and ensures that the analysis of the element can be performed in isolation.

Considering a design load, $N_{ED,fi}$, of 2,8 MN for the most loaded column, as evaluated in ISO/TR 24679-6, a fire design variable action combination factor Ψ_{fi} = 0,60, and evaluating the load ratio $\chi = Q_k / (Q_k + G_k)$ as 0,40, the characteristic value of the permanent load, P_{Gk}, and imposed load, P_{Qk}, on the column are evaluated using Formula (2):

$$N_{ED,fi} = P_{Gk} + \psi_{fi} P_{Qk} = P_{Gk}\left(1 + \psi_{fi}\frac{\chi}{1-\chi}\right) \quad (2)$$

where

P_{Gk} = 2 000 kN

P_{Qk} = 1 333 kN

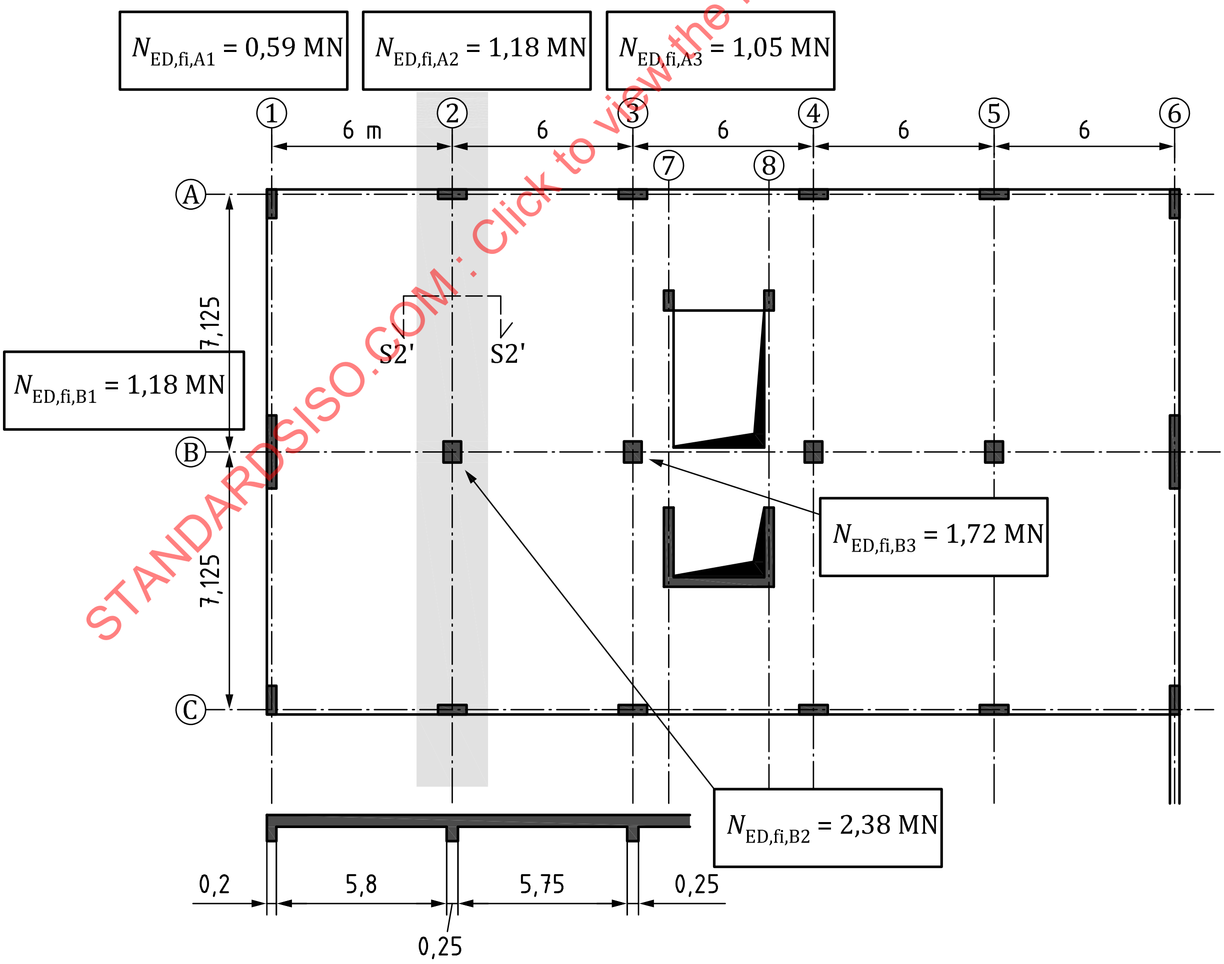


Figure 5 — Total design loads on concrete columns above

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

In agreement with ISO/TR 24679-6, the objectives of structural fire safety in this document are:

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

— conservation of property and continuity of operation.

Considering that the stability of the columns is crucial for the overall performance of the structure, and that columns in the ground floor are the most heavily loaded and have the greatest length, the functional requirement is:

— no loss of stability for the most loaded ground floor column, considering 240 min of ISO 834 standard fire exposure.

This functional requirement considers the ISO 834 standard fire exposure as a common point of reference in structural fire engineering. This functional requirement does not explicitly consider the cooling phase of realistic fires, although cooling phase performance is known to be a key issue[6]. Instead, it addresses cooling phase performance through a deemed-to-be-conservative assessment of the fire severity (ISO 834 standard heating regime duration); see also 5.4.

Imperfections in the column geometry and stochastic variations in the concrete compressive strength, steel yield strength and concrete cover imply uncertainty with respect to the column performance. Accounting for this uncertainty, the performance criterion selected to fulfil the above objectives and functional requirements, and confirmed through stakeholder consultation, is:

— structural stability of the most loaded ground floor column, considering 240 min of ISO 834 standard fire, is to be maintained with a reliability, p_s, of 99,5 % (i.e. a probability of failure given the fire exposure of no more than 0,5 %, i.e. $p_f \leq 5 \cdot 10^{-3}$).

5.3 STEP 3: Trial plan for fire safety of structures

The concrete building considered here has been designed for the ambient temperature. Achievement of the fire safety objectives with regard to structural fire resistance does not rely on any additional passive or active measures.

5.4 STEP 4: Design fire scenarios and design fires

The design fire has (hypothetically) been set at 240 min of exposure to the ISO 834 standard fire.

Achieving the performance requirement of maintaining stability for the most loaded column with 99,5 % reliability with this conventional fire exposure is deemed appropriate for ensuring the structural performance when exposed to any (reasonably possible) real fire development.

In other words, all possible design fire scenarios are aggregated into a single conventional design fire for ease of communication.

As noted in Clause 4, analyses whereby the fire exposure itself is also evaluated probabilistically are possible. The conceptual benefits of this more elaborate approach have to be weighed against the additional computational costs and complexity (both in terms of the calculation and the communication of the results).

5.5 STEP 5: Thermal response of the structures

The thermal response of the concrete column is evaluated deterministically considering the following thermal and physical properties, based on Eurocode 2:[5]

— water content: 2,0 % of mass;

— concrete density: 2 400 kg/m^3;

— thermal conductivity: average between the upper and lower limit given in Eurocode 2[5];

— siliceous aggregates;

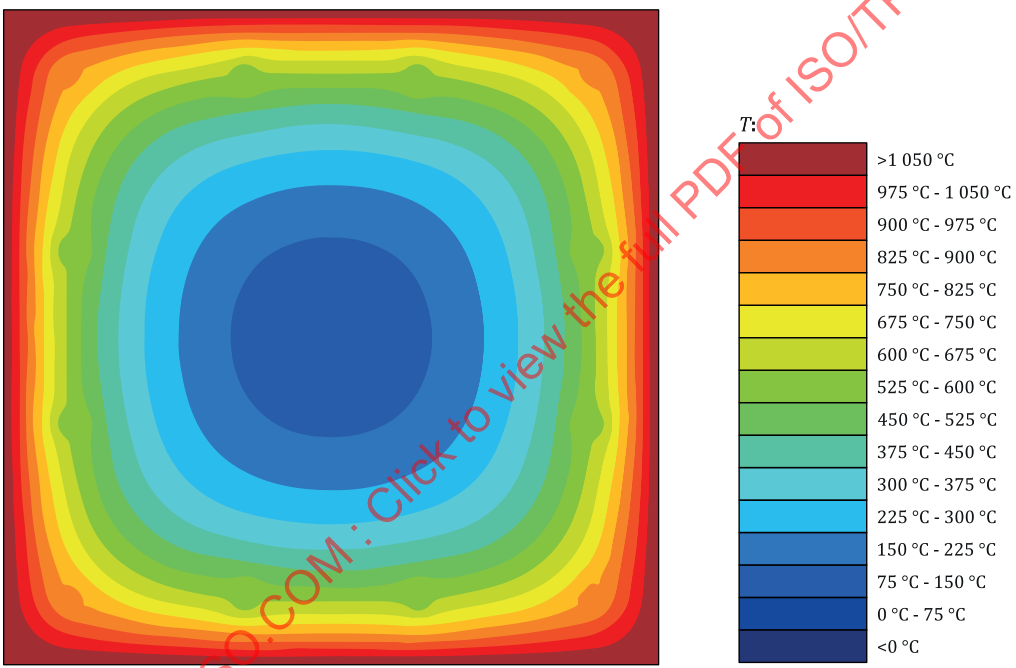
— emissivity related to the concrete surface: 0,7;

— the coefficient of heat transfer by convection is taken as 25 W/(m^2·K) (on all 4 sides);

— initial temperature: 20 °C.

With respect to the thermal response, a probabilistic analysis can also be applied. In the current evaluation, the uncertainty regarding the thermal properties of the concrete column is aggregated with the uncertainty on the fire exposure, both of which are taken into account through the deterministic ISO 834 standard fire exposure in accordance with Reference [5].

The heat transfer to the column has been calculated using an advanced model (see Reference [7]). The temperature distribution at 240 min of exposure is visualized in Figure 6.



Key

T temperature

TEMPERATURE PLOT

TIME: 14 400 s

Figure 6 — Temperature in column cross-section, at 4 h of ISO 834 standard fire exposure

5.6 STEP 6: Mechanical response of the structures

5.6.1 Structural model

The column is modelled in Reference [7].

In ISO/TR 24679-6, a 3D model was considered with the columns clamped at the bottom and with lateral and rotational restraint by the remainder of the structure experienced at their connection to the floor slab. As the column is considered in isolation here, a 2D analysis is performed. The column is modelled with a pinned support at the bottom, considering that there are 2 basement levels underneath the ground floor, and a roller support at the top (lateral restraint, but no rotational restraint). Neglecting rotational restraint results in a more onerous evaluation as rotational restraint helps reduce the lateral deformation of the column, but allows for the column to be modelled in isolation. Building system effects are deemed to be accounted for through the onerous assessment of the most loaded column.

The 4 m long column is modelled using 10 beam elements. As listed in ISO/TR 24679-6, the structural fire analysis considers the following points:[7]

a) the Bernoulli Hypothesis;

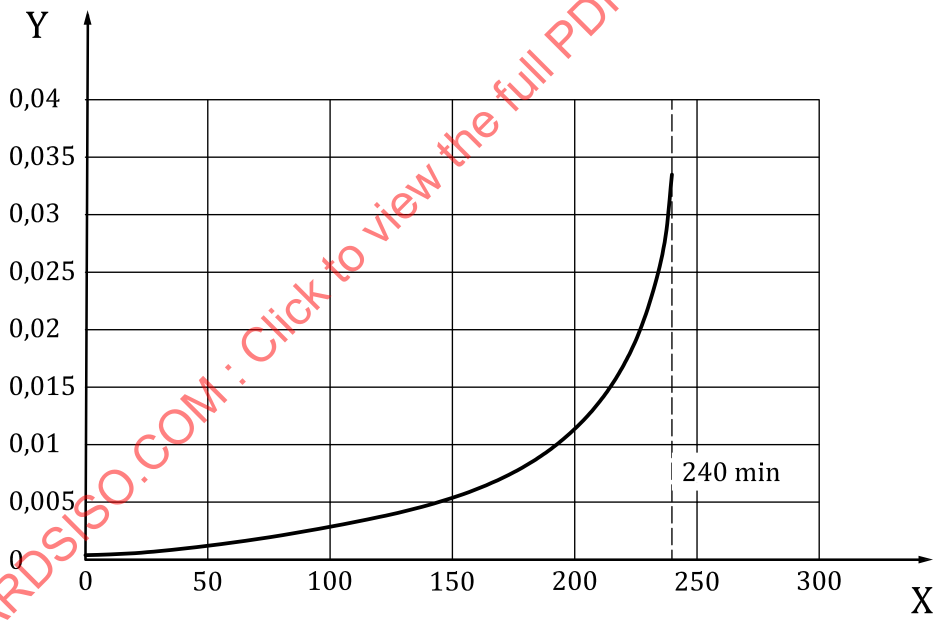
b) effects of non-uniform temperature distribution in the section (this is considered through a fibre model);

c) fracture energy (however, shear energy of the plane sections in the finite elements is ignored);

d) plastifications (in the longitudinal direction of beam elements only, meaning that uniaxial constitutive models are used in the beam element);

e) large displacements (however, strains are assumed to be small).

For a given realization of the column characteristics (concrete compressive strength, steel yield stress, steel yield stress retention (reduction) factor and geometry, see 5.6.2) and axial load, *P*, the deformation of the column and the time of structural failure are evaluated; see Figure 7, for example. The time of structural failure is evaluated considering a resolution of 1 min.



Key

X ISO 834 standard fire duration (min)

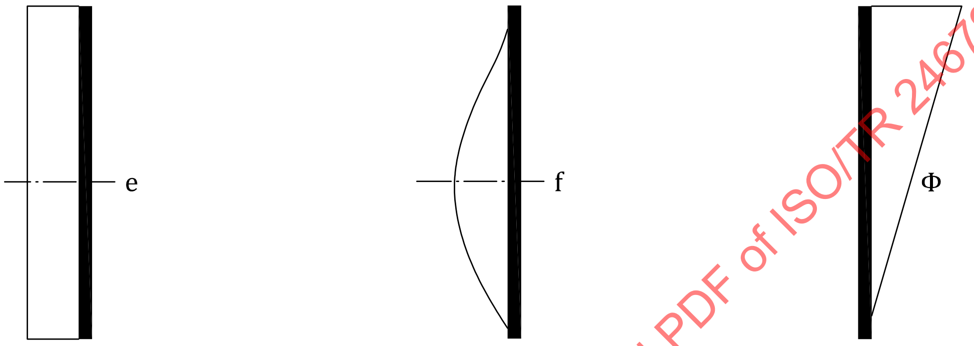
Y mid-height lateral displacement (m)

Figure 7 — Example of mid-height lateral deflection of a column according to the ISO 834 standard fire duration, and indication of assessed time of structural failure

5.6.2 Stochastic variables

The performance criterion requires the column to have a high reliability of maintaining structural stability up to 240 min of exposure to the ISO 834 standard fire. The achieved reliability is evaluated by taking into account the uncertainties associated with the characteristics (parameters) defining the column. The considered stochastic parameters and their probabilistic models are listed in Table 4. The meaning of the parameters eccentricity, out-of-straightness and out-of-plumbness are illustrated in Figure 5 and are as listed in Reference [8].

The steel yield strength retention factor at elevated temperatures is modelled in accordance with the probabilistic model specified by Khorasani et al.,[10] while the concrete compressive strength retention factor and concrete cover are considered deterministically through their nominal values. The nominal value of the concrete compressive strength retention factor is taken as specified in EN 1992-1-2.[5] However, recent research by Qureshi et al.[11] indicates that the variation in the concrete compressive strength retention factor can have an important effect on the capacity of fire-exposed concrete columns. The current understanding of the influence of this effect is indicated in 5.7. In the current evaluation, a compensation is implicitly incorporated through the considered model uncertainty for the resistance effect, K_R. On the other hand, the concrete cover variation does not markedly influence the considered column's failure probability.



a) Average eccentricity, *e* b) Out-of-straightness, *f* c) Out-of-plumbness, *Φ*

Figure 8 — The three basic eccentricities[8]

Table 4 — Probabilistic description of concrete column parameters, based on References [8], [9] and [10]

Parameter	Distribution	Mean μ	Standard deviation σ
20 °C concrete compressive strength (MPa)	Lognormal	$\frac{f_{ck}}{1-2V}=42,9$ (f_{ck}= 30 MPa)	$V{\cdot}\mu$ (coefficient of variation, V = 0,15)
20 °C reinforcement yield stress (MPa)	Lognormal	560 (f_{yk}= 500 MPa)	30
Steel yield stress retention (reduction) factor at elevated temperature (-)	Logistic	Temperature-dependent[10]	Temperature-dependent[10]
Average eccentricity, *e* (m)	Normal	0	$\frac{L}{1\,000}=0,004$
Out-of-straightness, *f* (m)	Normal	0	$\frac{L}{1\,000}=0,004$
Out-of-plumbness, *Φ* (rad)	Normal	0	0,001 5

The column characteristics exhibit uncertainty (e.g. out-of-plumbness, concrete compressive strength), and the load effects G and Q which coincide with a (nominal) fire event are uncertain. Furthermore, both the resistance effect, R, of the column load bearing capacity and the load effect, E, are associated with model uncertainties (K_R and K_E). The corresponding probabilistic models are listed in Table 5. For the permanent and imposed load effect, these models relate to the arbitrary point-in-time loads to be considered in conjunction with fire exposure.

The model uncertainties applicable for structural fire design are not yet well-established. In Table 5, the model uncertainty for the load effect is equal to the model uncertainty listed for normal design conditions. For a simply supported column (as is the case here) internal thermal restraint is taken into account through the resistance model. For the resistance effect, the model uncertainty in Table 5 is based on the model uncertainty for normal design conditions, under the additional consideration that structural fire assessments are likely subject to more variability (larger coefficient of variation, V) and a reduced conservative bias (lower mean value, μ). The model uncertainty also accounts for effects which are not taken into account directly, here: the variability of concrete cover and the concrete compressive strength retention factor. Model uncertainties currently remain an active area of research (see Reference [13], for example). In the absence of international guidance on model uncertainties, it is up to the designer to make an assessment which is justifiable under scrutiny.

Table 5 — Probabilistic description of the load and model uncertainty, based on References [8], [9] and [12]

Parameter	Distribution	Mean μ	Standard deviation σ
Permanent load, P_G (kN)	Normal	P_{Gk}	$V{\cdot}\mu$ (coefficient of variation, V = 0,10)
Imposed load, P_Q (kN)	Gumbel (5-year reference)	0,2 P_{Qk}	$V{\cdot}\mu$ (coefficient of variation, V = 1,10)
Model uncertainty for the load effect, K_E (-)	Lognormal (LN)	1,0	0,1 (coefficient of variation, V = 0,10)
Model uncertainty for the resistance effect, K_R (-)	Lognormal (LN)	1,0	0,15 (coefficient of variation, V = 0,15)

5.6.3 Probabilistic evaluation of the column load bearing capacity

The criterion of the column maintaining stability is given by Formula (3), where P_{max} is the load bearing capacity of the column and P_T is the total axial load. Considering the design fire specified in 5.4, P_{max} refers to the load bearing capacity at 240 min of ISO 834 standard fire exposure.

$$Z = P_{max} - P_T \geq 0 \tag{3}$$

The column capacity P_{max} refers to the 'actual' load bearing capacity of the column and P_T refers to the 'actual' load effect. When assessed through a model, appropriate model uncertainties (as listed in Table 5) need to be taken into account.

The distribution of P_{max} can be evaluated considering the distributions of the input variables in Table 4 and Table 5. This can be achieved either through a direct calculation (i.e. 'user-calculated analysis'), or by applying a 'listed' fragility curve. Alternatively, a semi-probabilistic approach can be used in case the type of distribution describing P_{max} (or a derived parameter) is known.

The above alternative approaches are listed below as separate example assessments in 5.7.

5.7 STEP 7: Assessment against the fire safety objectives

5.7.1 Example assessment 1: full probabilistic analysis — user-calculated analysis

5.7.1.1 Introduction

A first assessment procedure considers the direct calculation of the distributions for P_{max} and P_T in Formula (3).

The direct calculation of P_{max} is computationally expensive, taking into account repeated sampling of the numerical model and an iterative procedure for evaluating P_{max} (see 5.7.1.2). The total calculation time is at the time of publication in the order 2 000 core hours on a state-of-the-art laptop.

5.7.1.2 Evaluation of P_{max}

The column load bearing capacity P_{max} is evaluated numerically, using the Finite Element code SAFIR®[7]1), as elaborated in 5.6.1, and taking into account the model uncertainty K_R as listed in Table 5. In other words, P_{max} is evaluated by Formula (4), where $P_{max,num}$ is the direct result of the numerical evaluation.

$$P_{max} = K_R P_{max,num} \quad (4)$$

For a given column realization (i.e. for a given concrete compressive strength), the corresponding realization of $P_{max,num}$ is determined through the finite element code by evaluating the maximum load for which the run-off failure (illustrated in Figure 7) does not occur prior to the considered standard fire exposure time (in this case, 240 min).

The distribution of $P_{max,num}$ is evaluated using repeated (Monte Carlo type) random sampling (specifically: Latin Hypercube Sampling[14]), i.e. evaluating $P_{max,num}$ for different random realizations of the input variables listed in Table 4 through the numerical model. In total, 10^4 repetitions are performed. Latin Hypercube Sampling ensures that the full range of the input variables' distribution is sampled. For the applied number of realizations, the advantage of Latin Hypercube Sampling over standard crude Monte Carlo random sampling can be considered negligible. Results are visualized in Figure 9 (cumulative density function, CDF, and complementary cumulative density function, cCDF) and Figure 10 (probability density function, PDF), together with a lognormal approximation. The observed mean and standard deviation for $P_{max,num}$ are listed in Table 6. More computationally efficient, but approximate, methods exist for assessing the distribution of $P_{max,num}$ (see Reference [12], for example).

For the specific considered case, a lognormal approximation accurately describes the distribution of $P_{max,num}$. As both K_R and $P_{max,num}$ can be described by a lognormal distribution, their product P_{max} is described by a lognormal distribution as well, with mean and standard deviation as listed in Table 6.

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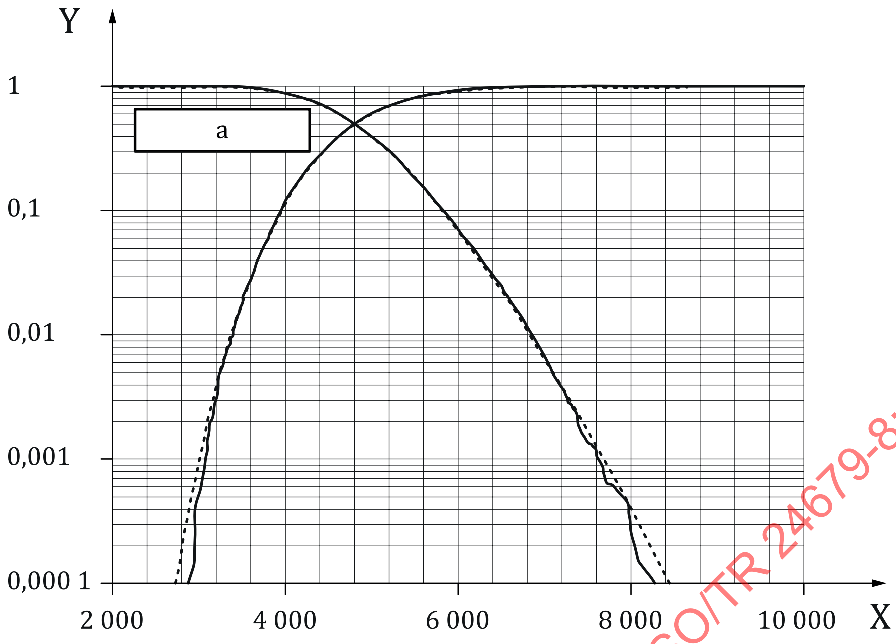
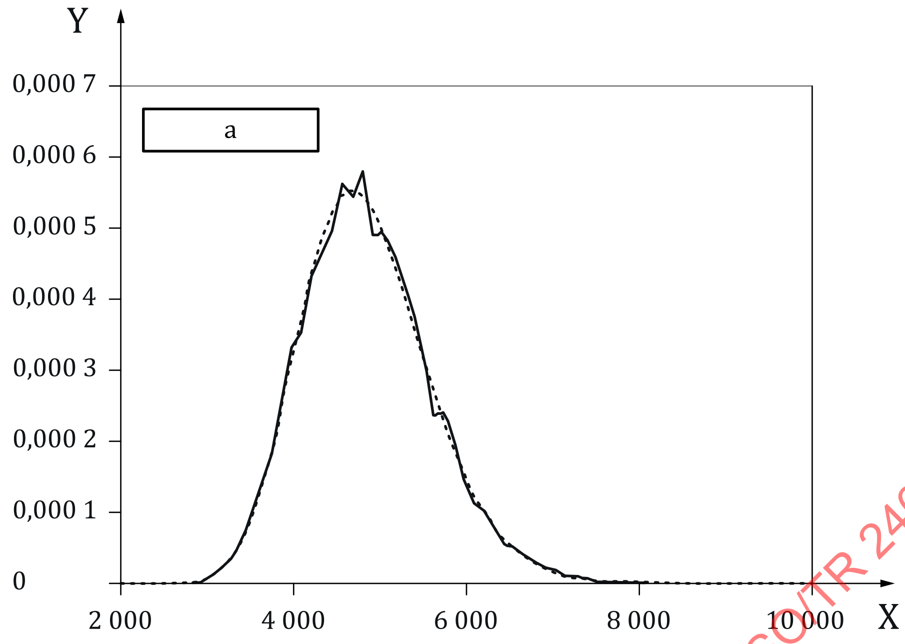


Key

Y	CDF (-); cCDF (-)
X	$P_{max,num}$ (kN)
a	240 min ISO 834
———	Monte Carlo Simulation (MCS)
--------	Lognormal (LN) approx.

Figure 9 — Cumulative density function (CDF) and complementary cumulative density function (cCDF) of $P_{max,num}$: numerical evaluation and lognormal approximation



Key

Y PDF (-)

X $P_{max,num}$ (kN)

a 240 min ISO 834

——— MCS

-------- LN approx.

Figure 10 — Probability density function (PDF) of $P_{max,num}$: numerical evaluation and lognormal approximation

Table 6 — Distributions describing P_{max}

Parameter	Distribution	Mean μ	Standard deviation σ	Coefficient of variation V
Numerical evaluation of the maximum load, $P_{max,num}$	Lognormal	4 854,1 kN	742,5 kN	0,15
Model uncertainty for the resistance effect, K_R (see Table 5)	Lognormal	1,0	0,15 (coefficient of variation, V = 0,15)	0,15
Column load bearing capacity, P_{max}	Lognormal	4 854,1 kN	1 045,9 kN	0,22

5.7.1.3 Evaluation of P_T

The total load effect, P_T, is evaluated directly through the distributions listed in Table 5 as:

$$P_T = K_E \left(P_G + P_Q \right) \quad (5)$$

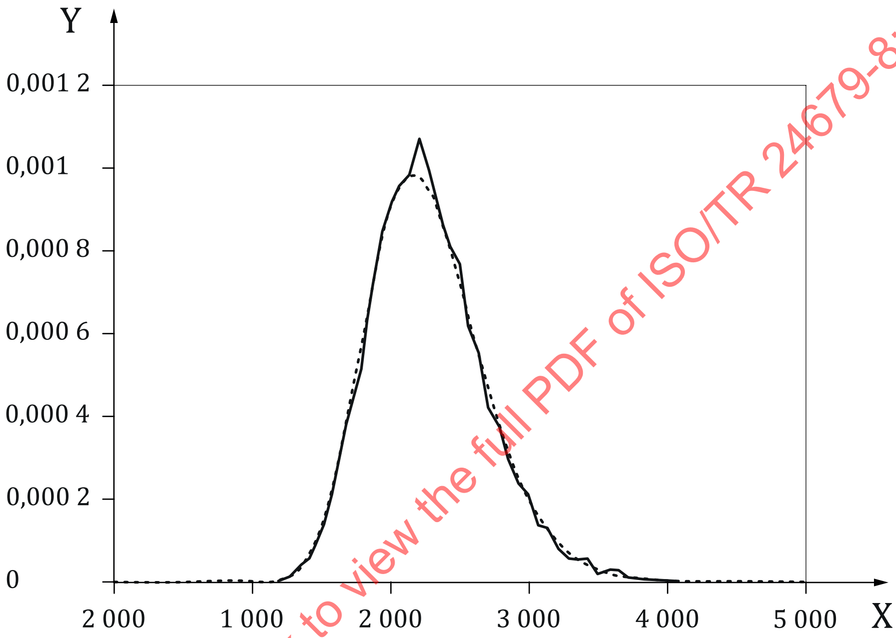
The total load can be approximated by a lognormal distribution, with mean and standard deviation assessed by a Taylor approximation, i.e. Formulae (6) and (7). For reference, the appropriateness of this approximation is illustrated in Figure 11. Considering Formulae (6) and (7), the coefficient of variation $V(P_T)$ is approximately 0,19.

$$\mu(P_T)=\mu(K_E)\left(\mu(P_G)+\mu(P_Q)\right)=2\ 266{,}6 \tag{6}$$

where $\mu(P_T)$ is expressed in kN.

$$\sigma(P_T)=\sqrt{\sigma_{K_E}^2\left(\mu(P_G)+\mu(P_Q)\right)^2+\mu_{K_E}^2\left(\sigma_{P_G}^2+\sigma_{P_Q}^2\right)}=421{,}2 \tag{7}$$

where $\sigma(P_T)$ is expressed in kN.



Key

Y PDF (-)

X P_T (kN)

——— MCS

-------- LN approx.

Figure 11 — Total load effect P_E: numerical evaluation and lognormal approximation

5.7.1.4 Evaluation of the reliability/failure probability

The reliability is evaluated in accordance with Formula (8), and the failure probability as its converse, as shown in Formula (9):

$$p_s=P[Z=P_{max}-P_T\geq 0] \tag{8}$$

$$p_f=P[Z=P_{max}-P_T<0] \tag{9}$$

where $P[.]$ is the probability operator.

As P_{max} and P_T are both described by a lognormal distribution, p_f is directly given by Formula (10), where Φ is the standard normal cumulative distribution function.

$$p_f = \Phi\left[\frac{\ln\left(\frac{\mu(P_T)}{\mu(P_{max})}\right)\sqrt{\frac{V_{P_{max}}^2+1}{V_{P_T}^2+1}}}{\sqrt{\ln\left(\left(V_{P_{max}}^2+1\right)\left(V_{P_T}^2+1\right)\right)}}\right]=3,6,10^{-3} \tag{10}$$

Considering Formula (10), the probability of failure given the standard fire exposure is lower than the performance criterion of $p_f \leq 5{\cdot}10^{-3}$. Equivalently, the reliability of the column is assessed to exceed 99,5 % for the prescribed standard fire exposure duration.

The design is accepted with respect to the structural fire safety objectives, in accordance with 5.2.

NOTE When considering the uncertainty with respect to the concrete retention factor at elevated temperature in accordance with Qureshi et al.,[11] a higher failure probability is found. However, a more detailed modelling also influences the model uncertainty assessment. In this respect, further developments in the state-of-the-art are to be monitored.

5.7.2 Example assessment 2: full probabilistic analysis — listed fragility curve

5.7.2.1 Introduction

This assessment procedure considers the application of listed fragility curves for P_{max}.

Fragility curves show the probability of exceeding a specified limit state according to a demand (load) parameter. Fragility curves are widely used in earthquake engineering, where they indicate the probability of exceeding a specified damage level according to a demand variable of interest, such as the earthquake intensity itself, or an intermediate response parameter such as inter-storey-drift.

Fragility curves can be listed in literature, by manufacturers or industry organizations, and allow for a fast reliability-based evaluation of the adequacy of the fire safety of the structure with respect to the considered performance requirement. The development of fragility curves can be computationally expensive, but can be done on a general basis and increases uniformity across projects.

In the following subclauses a fragility curve is applied which indicates the probability of exceeding the capacity limit state of the column, according to the applied load, P_{max}. This P_{max} thus corresponds with the distribution (cumulative density function) of the column capacity. Listed fragility curves (i.e. distributions) for P_{max} allow for the fast and easy application of dedicated reliability software, such as COMREL®[15][2], limiting computational cost.

COMREL® is a commercial package for structural reliability analysis. It incorporates a large number of different reliability methods and has ample validation. Other packages (including open-source software) exist. The user should be familiar with the reliability method applied and the validation level of the software.

5.7.2.2 Evaluation of P_{max} through listed fragility curve

In the hypothesis, P_{max} is listed for the given standard fire exposure and column design in scientific literature, or by a manufacturer or industry organization.

The fragility curve is visualized in Figure 12, together with its distributional definition. This definition corresponds with the result obtained through computationally expensive simulation in Table 6, after rounding.

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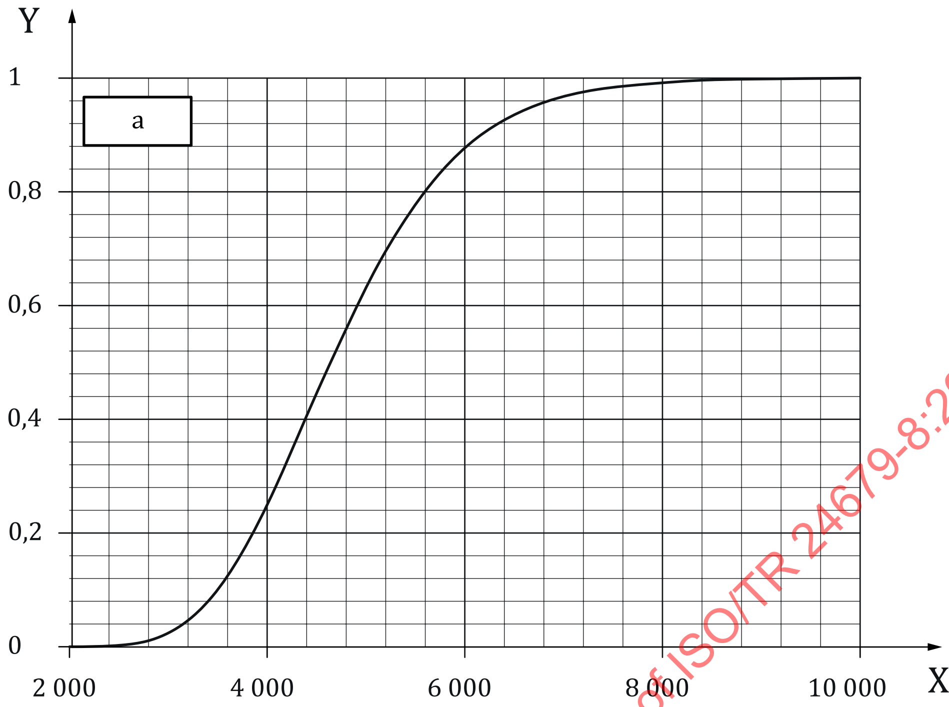

Key

Y	CDF (-)
X	P_{max} (kN)
a	ISO 834
———	240 min
--------	LN; (μ, V) = (4 850 kN; 0,22)

Figure 12 — P_{max} listed fragility curve

5.7.2.3 Evaluation of the reliability/failure probability

Taking into account the model for the load effect as listed in Formula (5), the limit state function for the column stability is given by Formula (11). The distributions for the parameters are reprinted for clarity in Table 7, taking into account Figure 12 and Table 5.

$$Z = P_{max} - K_E \left(P_G + P_Q \right) \tag{11}$$

Table 7 — Probabilistic models for the variables in Formula (11)

Parameter	Distribution	Mean μ	Standard deviation σ	Coefficient of variation V
Permanent load, P_G (kN)	Normal	P_{Gk}	$V{\cdot}\mu$	0,10
Imposed load, P_Q (kN)	Gumbel (5-year reference)	0,2 P_{Qk}	$V{\cdot}\mu$	1,10
Model uncertainty for the load effect, K_E (-)	Lognormal	1,0	$V{\cdot}\mu$	0,10
Column load bearing capacity, P_{max}	Lognormal	4 850 kN	1 067 kN	0,22

