RISK AND UNCERTAINTY IN CURRENT PRACTICE ON STRUCTURAL DESIGN FOR FIRE SAFETY

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ABSTRACT

During the last twenty years the concept of risk has been thoroughly exploited in several fire safety engineering applications. However, fire safety design of structures has remained fairly consequence-based and only implicitly considers the impact of the scenario frequency. Thus, the possibilities to apply an active approach to ensure that the temperature does not reach a level that will cause mechanical distress to the structure are limited. Current design practice results in an unknown level of risk and a switch towards a probabilistic approach would be beneficial to designers as well as regulators. This paper reviews the current design practice on fire safety of structures with focus on risk and uncertainty in order to support the development of a probabilistic model code on fire safety design of structures. Practical implications with current practice are identified and necessary steps that will support the future development of a model code will be proposed.

INTRODUCTION

During the last twenty years, the concept of risk, i.e. the frequency and consequence of a specific scenario set, has been thoroughly investigated in several fire safety engineering applications[1,2]. However, fire safety design of structures has remained fairly consequence-based, and only implicitly considers the impact of the scenario frequency. The CIB W14 undertook pioneering work in the 1980s when publishing their probabilistic design guide on structural fire safety[3] and their definition of the probability that a fire-exposed structure or structural member fails is now prevalent throughout the industry:

$$P_{failure} = P_{failure|\,flashover} \cdot P_{flashover|\,fire} \cdot P_{fire} \qquad [1]$$

Thus, for a structure to fail due to fire, first an ignition is required, secondly a flashover must take place given ignition, and finally, given the flashover, failure must occur. A tolerable value on the probability of failure could be met by reducing the probability that a fire occurs, by reducing the probability that a flashover occurs once a fire has started or by reducing the probability of a structural failure in the case of a post-flashover fire. Traditionally, structural fire safety design has been primarily concerned with the design of structures exposed to flashover fires. But, by using Equation [1] it is evident that either active or passive fire safety measures or a combination of the two, could be used to achieve a tolerable risk of failure. Equation [1], does however, give an incomplete picture of the prerequisites for structural failure as collapse could occur without flashover if the structural members are exposed to a localised fire. Such incidents are often ignored in the theories related to probabilistic structural design.

Over the last 10 years additional documents have been published on reliability-based design such as the ISO 2394[4] as well as Eurocode 1991-1-7[5]. These standards present necessary methods, principles and data related to probabilistic design and its calculation procedures for ambient loads and details on practical application regarding fire safety is limited. Thus, the CIB design guide needed further development in order to be used in practice and in the 1990s, a European research initiative entitled Natural Fire Safety Concept[6] tried to establish a more realistic and more credible approach to analysis of structural safety in case of fire that takes account of active fire fighting measures and real fire characteristics. The work resulted in a methodology to adjust the design value of the fire load considering danger of fire activation due to size of fire compartment and type of occupancy, as well as different active fire fighting measures (e.g. sprinklers, detection and a fire brigade). The methodology is a part of Appendix E of Eurocode 1991-1-2[7], but several European countries made reservations against its practical use.

Buchanan[8] calls for a new knowledge on both the nature of severe fires as well as on the structural behaviour in such fires. Buchanan states that quantitative risk assessment for structural safety will add a new dimension to solve some hard issues in design. Even though the scientific background on the application of reliability-based structural design is somewhat limited, practitioners are keen on adopting methods that optimizes designs and reduce construction costs and it is of interest to regulators that the safety level is well known and understood. Thus there is a need to bridge the gap between theory and practical application by developing tools that will utilize the concept of risk and treat uncertainties related to structural design for fire safety. This paper aims at reviewing the current design practice on fire safety of structures with focus on risk and uncertainty with the objective to support the development of a probabilistic model code on fire safety design of structures. A literature review on both structural design for ambient conditions and fire will form the basis of the paper. The discussion that follows will raise key issues as well as propose a path forward that will support the future development of a model code.

CURRENT DESIGN PRACTICE OF STRUCTURES

Structural design should consider a variety of actions such as permanent actions, variable actions and accidental actions. Both permanent actions and variable actions relate to ambient conditions and they are present at all times. Accidental actions are actions, difficult to define in terms of intensity and frequency, which may occur as a result of accident or exceptional circumstance. Examples of accidental actions include explosions, impacts (collisions) and fires. Design procedures for ambient actions are very similar to those applied for accidental actions related to impact, explosion, earthquake, etc. A review of current design practice related to both ambient actions and fire is of interest in order to identify its strengths and weaknesses.

Actions on structures related to ambient loads

The requirements on structures are based on target reliabilities and are differentiated depending on the severity of a collapse. Current practice is defined as semi-probabilistic with the use of characteristic values on loads and specific partial factors. The partial factors depend on the probability of an unwanted deviation from the characteristic value, the inaccuracy of the calculation model and the consequences of collapse. Partial factors are thus an approach used to deal with uncertainty and variability as a result of unknown failure mechanisms, imperfect theory, higher loads, inaccurate material properties and human error. The ultimate limit state of the structure is evaluated for a number of load combinations, which all must meet the design criteria, i.e. the resistance, R, must be greater than the stress, S, applied to the structure:

$$R - S > 0 \qquad [2]$$

The partial factor method is one of two procedures recognised in the ISO 2394 document on Reliability of Structures[4] for the verification of structural reliability. The other procedure, known as “the full probabilistic method”, uses defined distributions for load and resistance and evaluates the probability of collapse, $P_{(R-S<0)}$, in relation to a specified risk criteria, or target reliability, P_{target}. Equation [1] is then rewritten as shown below.

$$P_{(R-S<0)} \leq P_{target} \qquad [3]$$

The partial factor method is the most commonly used of these procedures as the method is supported by a set of action and material codes that provides the necessary information on calculation models, characteristic values, etc. However, efforts have recently been made by the Joint Committee on Structural Safety (JCSS) to develop a probabilistic model code[9], which deals with basis of design in terms of target reliabilities, interpretation of probabilities as well as development of probabilistic models for structural loads and resistance. Target reliabilities are linked to structural risk acceptance criteria and expressed either as the accepted minimum reliability, or as the accepted maximum failure probability, P_f. Commonly, a generalised reliability index, β is chosen as the reliability measure:

$$\beta = -\Phi^{-1}\left(P_f\right) \quad [4]$$

In Equation [4] the inverse standardised normal distribution (Φ^{-1}) is used to transfer the probability of failure into the reliability index. Target reliabilities are given in Eurocode EN 1990[10] and in ISO 2394[4] and they consider a maximum allowable individual risk of app. 10^{-6} per year as a reference. This reference value could be transformed to a maximum allowable probability of failure of the structure depending on the conditional probability of a person being killed, given the failure of the structure. The JCSS[9] states that the probability of structural failure should depend on the risk to life, economic consequences and type of failure. A target reliability index β of 4.2 ($P_f = 10^{-5}$) is considered as a central value for most design situation, but values between 3.1 ($P_f = 10^{-3}$) and 4.7 ($P_f = 10^{-6}$) could be used depending on the consequence class of the structure. A proposal to differentiate the required safety in the event of fire depending on people evacuation has been made[6]. The EN 1990 requirement on $1.3\cdot10^{-6}$ per year should be applied to buildings where escape is practically impossible (e.g. high rise buildings). For buildings with normal evacuation a target failure probability of $1.3\cdot10^{-4}$ and for buildings where escape is difficult (e.g. hospitals) a reference value of $1.3\cdot10^{-5}$ is proposed. Note that the target reliability is related to the system as designed, i.e. not built. Failure due to human error or ignorance and failures due to non-structural causes are not covered. Such failures are supposed to be covered by design review and other quality assurance procedures.

The definition of probability used by the JCSS is the Bayesian interpretation where probabilities are considered as the best possible expression if the degree of belief in the occurrence of a specific event. The Bayesian interpretation does not claim that the probabilities are direct and unbiased predictors of the frequency of collapse that could be observed in practice. Nevertheless, if the analysis is carried out thoroughly, the probabilities will be correct when averaged over a large number of design situations and it will be possible to derive design values based on common practice. However, the lack of statistical data may lead to uncertainties in statistical parameters and type of distribution.

Actions are divided into different types depending on their time variation, e.g. permanent, variable and accidental actions. Permanent actions are often the sum of many individual loads and they may be represented by a normal distribution. Variable actions do vary in time, but it is the magnitude of the largest extreme load that occurs during the specified reference period for which the probability of failure is calculated. Accidental actions are considered in the same manner as variable actions where the magnitude of the load is of greatest interest. The partial factor method uses characteristic values on actions that in combination with a specified partial factor will meet the target reliability. Characteristic values are often selected as the 50-year load, i.e. a load value that will not be exceeded by a probability of 98 % during a reference period of one year. The resistance of the structural member is treated in a similar, but opposite manner. The characteristic value the resistance is decided by the 5 % percentile derived from material testing. Both the characteristic value of the action, F_k, (stress) and the characteristic values of the resistance, R_k, are assigned partial coefficients, γ to treat uncertainty related to the variable. The design values (F_d and R_d) are defined as follows:

$$F_d = \gamma_{F_k} \cdot F_k \quad [5]$$

$$R_d = \gamma_{R_k} \cdot R_k \quad [6]$$

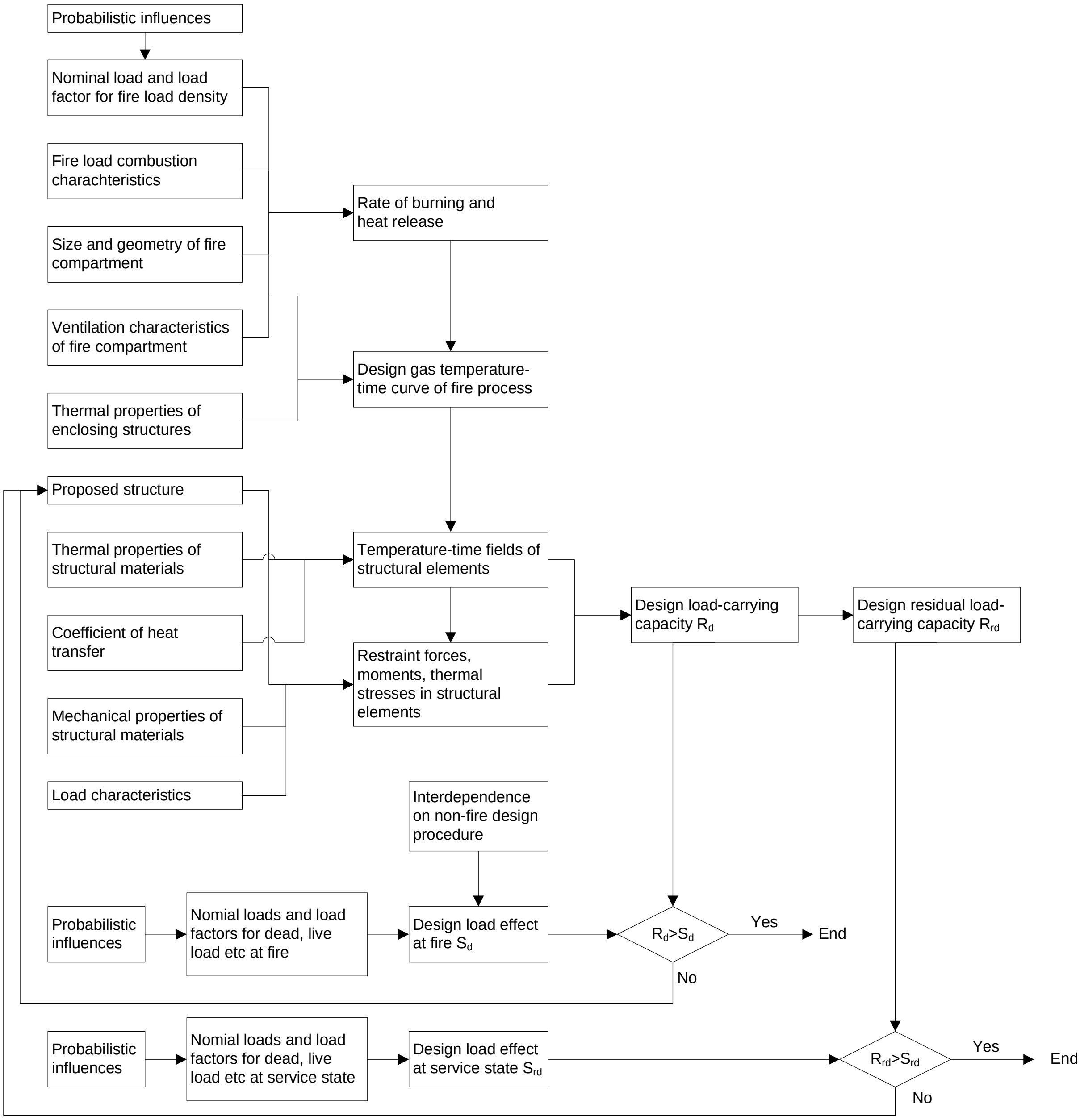
Actions on structures related to fire

Fire differs from the other accidental actions as a fire cannot be expressed in load-specific units (e.g. kN/m^2). Instead, fire is defined as an in-direct action causing a reduction of the load-bearing capacity of the structural member. Thus, fires cannot be treated as loads, even though they are categorised as such. Current practice is focused on expressing the thermal load that a fire has on a structural member and how this thermal load reduces its load-bearing capacity. Historically, the thermal load of a fire has its basis in the nominal or standard fire curves developed for fire resistance furnace tests of building materials and elements for their classification and verification. These curves are the simplest way to represent a fire by pre-defining some arbitrary temperature-time relationships, which are independent on ventilation and boundary conditions and they are often referred to as the prescriptive design approach. In the 1970s a new set of methods were developed which allowed for an analytical approach to the design of structural fire safety compared to the predominate use of standardised fire resistance tests in the prescriptive approach. This development made it possible to quantify thermal exposure based on the conditions of a fully developed fire determined by the combustion characteristics of the fire load, the ventilation of the fire compartment and the thermal properties of the enclosing structures. Today, most national legislation within the EU recognises both these approaches when designing structural fire safety.

The legislative requirements on the load-bearing capacity in the event of fire vary with service category and safety class[11]. Most buildings should have sufficient stability and load-bearing capacity during the entire fire sequence. In other buildings the time required for escape, rescue and preventing fire spread to adjoining constructions are setting the demands on the structure. Buildings assigned to the lowest safety class only require sufficient stability and load-bearing capacity for at least the time required for escape. There are basically two different set of approaches to be used when evaluating stability in the event of fire –the prescriptive and the analytical approach. The prescriptive approach is a classification based on the results from a standardised testing procedure according to e.g. EN 13501[12]. This approach, where the temperature-time response is determined by the standard, is often referred to as a "pseudo" fire as the temperature characteristics cannot be generated from basic principles. However, it is a possibility for the building industry to get a rating on a specific structure which simplifies the design process. The analytical approach is based on a "real" fire, called the parametric fire curve in the Eurocode[7]. The parametric fire curve can be derived by the use of basic principles considering the fire load, compartment geometry, material properties and available air supply. The temperature-time response is based on empirical curves which can be used to determine the thermal load on a structural element and the resulting reduction in load-bearing capacity.

Pettersson et.al[13] did pioneering work in the 1970s regarding fire safety design of structures and outlined a differentiated design procedure which was slightly modified by Magnusson[14] to illustrate probabilistic influences (see Figure 1). The modification made by Magnusson indicates how the variabilities of different components are lumped together in the resistance term *R* and the loading term *S*. The procedure in Figure 1 consists of three major models, i.e. the heat exposure model (*H*), the structural model (*S*) and the reliability model. The flow diagram provides insight to the various variables that constitute the fundamentals of current design practice. The assessment of the design gas temperature-time curve in Figure 1 is carried out with a heat exposure model and the CIB[3] presents a set of such models as well as models for structural response. The heat exposure model (H) will be used to determine the rise of temperature as a function of time and a structural response model (S) will be used to determine heat transfer to and within the structure and the resulting load bearing capacity of the structure when exposed to heat. There are three types of heat exposure; standard temperature-time response (H_1), time equivalence approach (H_2) and a real, natural fire (H_3). There are three structural models as well; individual members (S_1), sub-assemblies of members (S_2) and the complete structure (S_3). The majority of the design is carried out in the H_1/S_1 domain, and when additional flexibility is required, the H_2/S_1 or the H_3/S_1 domain is used. Purkiss[15] states that the assessment method used to evaluate fire performance is related to the heating or temperature exposure rather than the structural model, and consequently the use of S_2 and S_3 for structural assessment is limited both by the resources required and the uncertainties introduced.

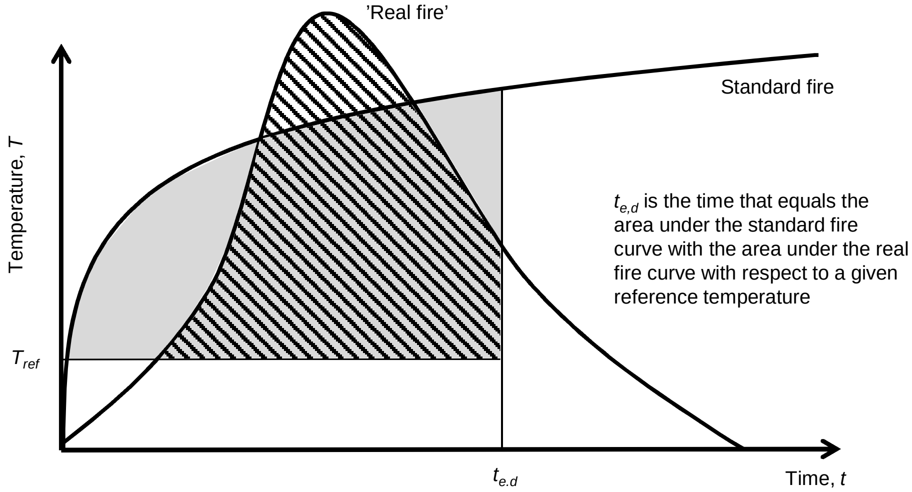
Figure 1. Flow diagram of fire safety design procedure of load-bearing structures[14].



In the standard fire test (H_1) a structural element (S_1) is loaded with an equivalent load related to the stress that the element would be exposed to when placed in a structure. The element is exposed to heat following a prescribed temperature-time curve until failure of the element occurs. The time when failure occurs is regarded as the fire rating of the structure (R 30, R 60, R90, etc.) The prescribed temperature-time curve is defined in EN 13501[12]. The second assessment method – time equivalence (H_2) is strongly linked to H_1, but allows for some considerations on the actual fire load, compartment geometry and construction material. Time equivalence can be shown either based on temperature or normalised heat load. The temperature-based time equivalence method provides a correlation between natural fire exposure and similar exposure in the standard fire test and can thus replace fire testing in some extent. Unlike the standard fire where the temperature is prescribed by the standard itself, the temperature of a natural fire (H_3) is a function of the compartment size, the type of compartment, available combustible material and air supply. EN 1991-1-2[7] provides empirical curves to express the temperature-time relationship of such fires and it is commonly assumed that a structural element should maintain its capacity during the complete fire duration when exposed to a natural fire.

Figure 2 illustrates these three heat exposure models. Note that H_2 'translates' the heat exposure from a real fire (H_3) to that of the standard fire test (H_1).

Figure 2. Available heat exposure models.



The introduction of ideas concerning possible trade-offs' between active and passive fire safety measures were described in the 1970s by Baldwin and Thomas[16]. The arguments behind such trade-offs are essential statistical where the active fire safety measure (e.g. sprinklers) reduced the likelihood of a severe fire and thus can the passive requirement on the structure be reduced as well. The Natural Fire Safety Concept[6] introduced a possibility to balance the passive requirement on the structure by modifying the design fire load to be applied when assessing the temperature-time relationship of a natural fire. Similar to ambient loads as described by Equation [5], the design fire load, $q_{f,d}$ is calculated by multiplying a partial factor with the characteristic fire load, $q_{f,k}$:

$$q_{f,d} = \gamma_{q_f} \cdot q_{f,k} \qquad [7]$$

According to the Natural Fire Safety Concept, the partial factor for the fire load is divided into sub coefficients to take into account the compartment size, γ_{q1}, the building type, γ_{q2}, and the different active fire fighting measures, γ_{ni}. Thus, the characteristic fire load has to be multiplied by the individual sub coefficients to obtain the design fire load:

$$\gamma_{q_f} = \gamma_{q_1} \cdot \gamma_{q_2} \cdot \gamma_{n_i} \qquad [8]$$

$$\gamma_{n_i} = \gamma_{n_1} \cdot \gamma_{q_2} \cdot \ldots \cdot \gamma_{q_{10}} \qquad [9]$$

The EN 1991-1-2[7] has apprehended this concept of adjusting the design fire load in its Annex E, which is considered to be informative and not mandatory for the EU member countries to ratify. The Swedish building regulations have, e.g. forbidden the use of the appendix due to some of its drawbacks. However, a partial factor of 0.61 could be applied to the characteristic fire load if the building is fitted with fire sprinklers. If a prescriptive design approach is followed a requirement of 90 min fire rating could be reduced to 60 min in a sprinklered building. He and Grubits[17] outline a methodology to be used when assessing risk equivalence between a sprinklered and a non-sprinklered building where the fire rating of the structure is calculated to fulfil the equivalence criterion. They suggest that their probability-based analytical approach for determining the residual fire resistance required when sprinklers are installed as a supplement to code requirements can be applied to other active safety measures as well. Although, the approach requires knowledge on random variables, failure criteria, probabilities of failure and that the consequences of failure is of similar nature.

IMPLICATIONS WITH CURRENT DESIGN PRACTICE

Naturally, there are several advantages with current design practice on structural fire safety and one of the major advantages is the simplicity of the methods. A construction element can be exposed to a standard fire in a testing facility and hence qualify to be used in a building. The available calculation models are somewhat calibrated to result in a similar design as the result of a fire test. However, during the last decade questions have been raised in scientific publications regarding the validity of the methods and the drawbacks related to them. This section will describe some practical implications with the current design practice.

The standard fire exposure forms the basis of most structural designs, even though it is considered unrepresentative of real fire conditions. Furthermore, when more realistic fire conditions are used, significant uncertainties with regard to the input parameters, in particular the magnitude of the fire load, are introduced and need to be treated. As the current design practice is mainly consequence-based it is not possible to explicitly consider risk, i.e. the combined effect of the frequency and the consequence of a specific scenario set. Thus, both the prescriptive and the analytical design approaches result in an unknown level of risk. This is an unwanted situation both for both designers and regulators. The designer asks for freedom regarding which method to use in order to fulfil the building requirements, and the regulator has an interest in a well-known safety level as well as a similar safety level within different types of buildings.

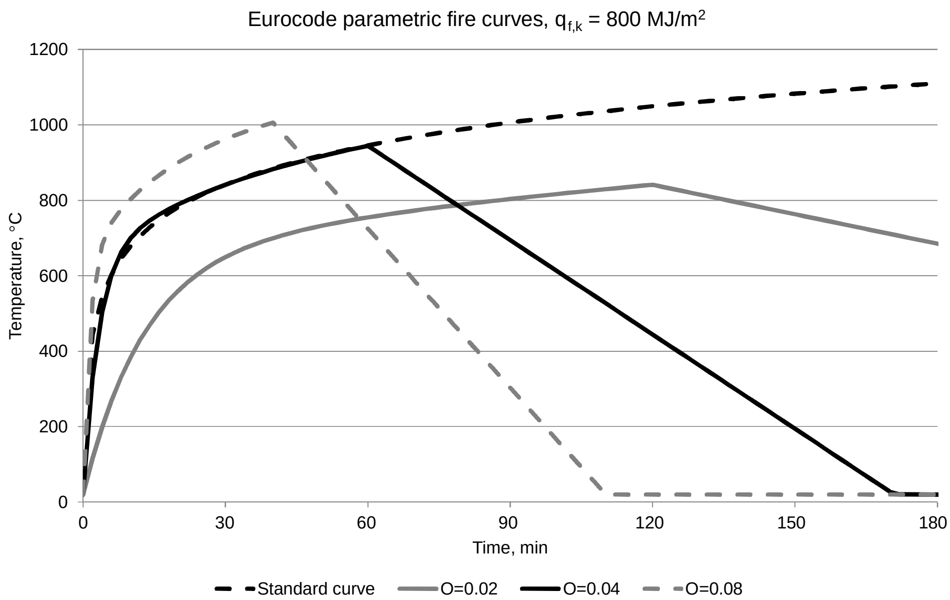
In the prescriptive approach, risk is mainly controlled as a function of building height as the requirements on the load-bearing capacity are increased with the height of the building. The analytical approach gives an opportunity for a more flexible design as some building characteristics such as the fire load density, the compartment size, the air supply and the construction material could be taken into consideration. Some building codes apply a safety factor (e.g. 1.5) to the design fire load for tall buildings implying that tall buildings need additional protection as they are exposed to potentially larger consequences. Designing for a larger fire load, will result in less probability that the fire duration exceeds the fire rating of the construction. If the design fire load is increased, the construction is exposed to a higher maximum temperature and longer fire duration as shown in Figure 5. Active fire safety measures, such as an automatic sprinkler system, do allow for a reduction in design fire load or the possibility to design for lower structural requirements when using the prescriptive approach. However, the possibilities to apply an active approach to ensure that the fire is contained or that the temperatures do not reach a level that will cause mechanical distress to the structure are limited.

Temperature-time relationship

One of the most influential parameters in fire safety design of the structural members is the gas temperature surrounding the members of construction, as this temperature will decide the load-bearing capacity of the member when exposed to fire conditions. Uncertainties related to this parameter will therefore have a strong influence on the safety level. The designer could choose a heat exposure model from a standard fire exposure, a time-equivalence concept, or a natural fire. All methods have several drawbacks that need to be considered. Purkiss[15] describes some inherent drawbacks of the standard fire test related to the very nature of the test and in part due to the uses of the tests. Standard fire testing is expensive and time-consuming. The data obtained from a test are only applicable to that specific test. Test specimens are limited to size as well as the load conditions are simplified in relation to real world conditions where, e.g., only axial loads are applied but the most critical situation for columns is when it is subjected to moments[15]. Interesting data on reproducibility has been published by Dotreppe et.al[18] where the same column maintained its load-bearing capacity for 84 min in one test and 138 min in another. Platt[19] summarises results from different sources on the actual performance of fire rated structures. E.g., the average value of the test failure time is app. 25 % higher that the specified fire rating of the structure. This is a logic consequence of the basis of the fire tests as the test often is interrupt when the desired fire rating is achieved. Thus, fire test do not provide information on the actual performance in the event of fire.

A natural fire is considered to have the best adaption to the specific conditions in the fire compartment. Nevertheless, several issues have been revoked on the application of such fires, e.g. the assumption of a uniform temperature distribution and the discrepancies between the fire curves and full scale tests. The temperature-time curve for a natural fire is assessed by using a one zone model of the fire compartment where the temperature is uniformly distributed within the room. Naturally, such an assumption has limited validity for large fire compartments and consequently the EN 1991-1-2[7] limits the use of the parametric fire to rooms with a floor area of maximum 500 m^2 and a ceiling height of maximum 4 m. However, full-scale testing for a smaller fire compartment at Dalmarnock[20] indicates that the 80% percentile of the temperature exceeds the mean value by 25 % and from the mean value. The time to reach the maximum value deviates in a similar fashion. Barnett[21] shows that the parametric fire deviates from experimental results and other computational models and Yii et.al.[22] illustrate the effect of fuel type and geometry of the compartment. It has been shown that the temperatures will be lower if the compartment has large openings to the outside or if it is well insulated. The result is the opposite for small openings and less insulation. Meanwhile, significant uncertainties relate to the actual opening conditions in the event of fire as the common assumption is that all doors and windows that do not have a fire rating are open. The dependence of the parametric fire curve to the opening factor is illustrated in Figure 3, where the thermal inertia of the construction is 1160 W $s^{0.5}$ / m^2 K. Note that the growth phase of the temperature-time curve for an opening factor of 0.04 $m^{0.5}$ is very similar to the standard fire.

Figure 3. The dependence of the parametric fire on the opening factor.

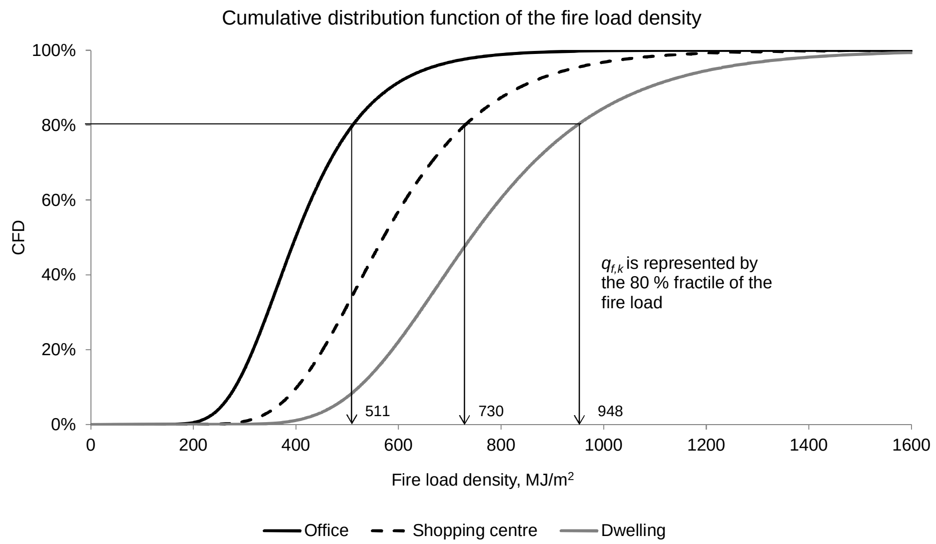


The parametric curve has its basis in the so called Swedish fire curves introduced by Magnusson & Thelandersson[23]. However, concerns have been raised on the validity of the initial assumptions made by Magnusson & Thelandersson on ventilation controlled fires with one vertical opening and wood-based fuel. Synthetic material has different burning behaviour[22] and has shown to produce higher temperatures. The presence of more than one opening in a real fire will result in a non-uniform temperature distribution as a large part of the combustion takes place in the vicinity of the openings. Feasey and Buchanan[24] propose some changes to the parametric fire in EN 1991-1-2 in order to get a better estimation of temperatures in post-flashover compartment fires. They also indicate a need for further research into post-flashover behaviour to deal with challenges related to e.g. fuel geometry and the presence of other ventilation openings apart from the limitation on one vertical opening introduced by Magnuson & Thelandersson. The decay rate of the parametric fire in EN 1991-1-2 is criticised by several authors for not following a natural cooling curve[21] and for being either too long or too short depending on the opening factor and the thermal properties[24]. Hertz[25] shows that a structural element of concrete could have its load-bearing capacity reduced by 50 % during the growth phase of the temperature-time curve and reduced by another 50 % during the cooling phase. Thus, depending on construction material, the cooling phase is of great importance to the performance of the structural element in the event of fire. Equation [1] in the introduction of this paper states that a fully-developed fire is required to put enough thermal stress to the structure to cause collapse. However, this assumption is questionable in larger compartments where flash-over is unlikely to occur and the temperature in the upper gas layer is non-uniform. For such compartments, Stern-Gottfried and Rein have introduced a concept of travelling fires[26,27]. They introduce ideas on near field and far field fire exposure and a fire that exposes structures to initial far field heating, near field heating as the fire travels by the structure and posterior far field heating.

Fire load data

Most fire load data is 40-50 years old and it is necessary to address the relevance of this data in relation to the fire loads today given the increasing use of synthetic materials in furnishing etc. Characteristic values on the fire load (80 % fractile) are given in EN 1991-1-2[7] for different groups of occupancies such as dwellings, hospitals, hotels, offices and shopping centres. EN 1991-1-2 also provides mean values of the fire load density and a statement that a Gumbel distribution is assumed for the fire load. Figure 4 illustrates the cumulative distribution of the fire load density in offices, shopping centres and dwellings. The distributions have been plotted by using data from EN 1991-1-2.

Figure 4. Cumulative distributions of the fire load density.



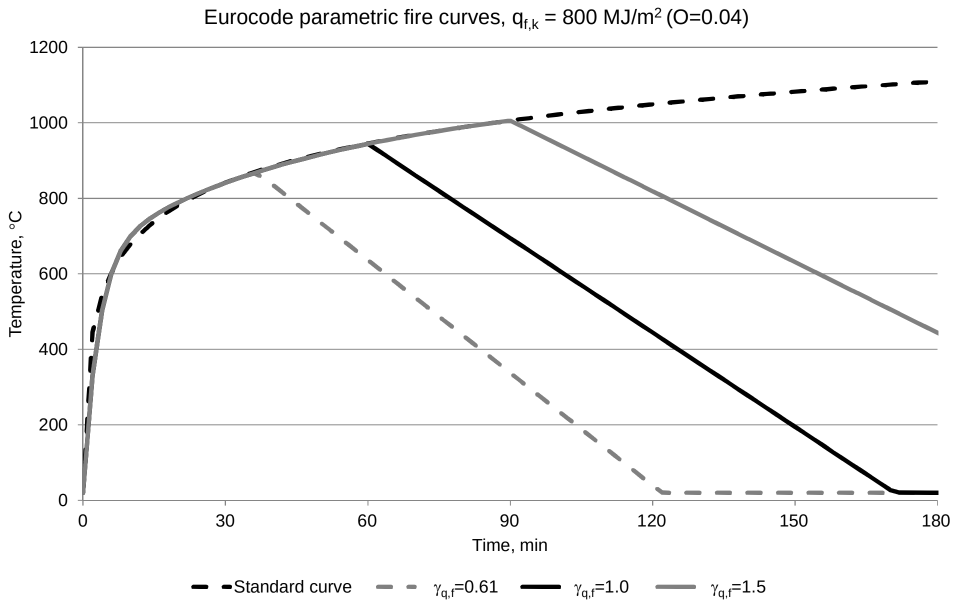
A fire load survey by Thauvoye et.al.[28] compare fire load density in Eurocode with recent surveys in shopping centres, hotels, hospitals and offices. It is concluded that data provided by EN 1991-1-2 on office buildings do not seem to be safe as the densities in the survey are app. 40 % higher. The same conclusion is drawn for shopping centres. Opposite conditions applies on fire load densities in hospitals and fire load densities in hotels show good agreement with Eurocode data. Thauvoye et.al.[28] conclude that wood material continue to be a great proportion of the fire load and plastics are only present in small quantities. The generalisation of fire load densities to represent wide-range occupancy groups, e.g. shopping centres, may result in an uncertainty whether the fire load is representative to a specific fire compartment within a the building. The average fire load in shopping centres is 600 MJ/m^2 according to EN 1991-1-2, which is similar to 571 MJ/m^2 presented in the Swiss study[28]. Although, when examining individual fire load densities presented by CIB[3] in a variety of occupancies within a shopping centre several type of occupancies are identified where the fire load exceeds the average value of the occupancy group.

E.g., book stores have a characteristic fire load density of app. 1250 MJ/m^2 and the probability that the actual fire load will exceed the characteristic value in EN 1991-1-2 is 83 %, compared to 20 % which should be the result due to the definition of the characteristic fire load as a fire load which is expected not to be exceeded during 80 % of time.

Use of active fire safety measures to protection structural members

Several national building regulations allow for a reduction in load-bearing capacity when buildings are fitted with sprinklers. But, other active safety measures such as smoke ventilation, fire brigade response, and oxygen depletion systems are not provided with equal opportunity. A key question is to decide on the residual level of passive fire safety applied to the structure that when combined with sprinklers will result in the same safety level as the code required fire rating. A possibility to reduce the design fire load will result in a construction that is designed for a lower maximum temperature and shorter fire duration, see Figure 5. In Figure 5, $\gamma_{q,f}$ represents the partial factor in Equation [7] and under normal circumstances it has a value of 1.0. If sprinklers are fitted in the building a value of $\gamma_{q,f}$ = 0.61 could be applied according to EN 1991-1-2. For illustrative purposes, the parametric fire for buildings taller than four stories[29] ($\gamma_{q,f}$ = 1.5) is shown in the figure.

Figure 5. The dependence of the parametric fire on the partial factor for the fire load.



Current practice balances the level of residual fire protection by allowing for a construction to be designed with less maximum temperature and shorter fire duration. Note that the rate of increase in the growth period is independent of the fire load and it could be questioned if this is an appropriate approach as fires in sprinklered buildings could have high, but short duration. Using $\gamma_{q,f}$ = 0.61 is although conservative compare to what has been proposed by the Natural fire Safety Concept[6] where a minimum value $\gamma_{q,f}$ = 0.13 could be applied the building has several active fire safety measures in place. Such a low value on $\gamma_{q,f}$ results in practically no residual fire protection at all as the temperature only reaches a maximum of 160 °C and a localised fire will definitely be more severe.

DISCUSSION

The conceptual approach on a probability based design guide on structural safety presented by CIB[3] points out that the only consistent method available to treat the uncertainties related to behaviour of structures in the event of fire is a probability based design approach. Some guidance on the use of a risk-based model is provided EN 1991-1-7[5] where it is stated that the risk of collapse could be assessed by the frequency of fire, the likelihood of a severe fire and the likelihood that the fire duration of a severe fire exceeds the capacity of the structure. The risk of collapse due to fire could then be evaluated towards an acceptable level of safety. Thus, well established target reliabilities are necessary to implement a probabilistic design code on structural safety. The target reliabilities must represent the current state of risk perception in the society and it should be investigated how aspects on both individual risk and societal risk ought to be considered. It would be preferable if acceptable levels of risk could be based only upon the probability of collapse as there are a number of large uncertainties related to the estimation of the consequences of collapse, e.g. regarding time to failure and the number of people left in the building. Ditlevsen[30] points out that a target reliability criterion must be accompanied by a reference to the specific code format by which it is defined as a constant reliability in one code does not imply constant reliability in another code. Therefore it is necessary to develop a model code in conjunction with the establishment of target reliabilities.

Design strategies for accidental loads in ISO 2394[4] allow for a reduction of the probability of the action, a reduction of the action intensity as well as a reduction of the effect of the action by limiting the amount of damage or making the structure strong enough. From a philosophical point of view, it should be arbitrary to the society whether the safety of a structure is based on a preventive or a protective approach. However, special attention must be given to the differences between active and passive fire safety features and their failure modes. Time to failure is of great interest and there are fundamental differences between e.g. sprinkler water being shut off and insufficient coating of structural members.

The review of the current design practice and its practical implications identified several areas related to the heat exposure model (see Figure 1) that need to be developed and considered in a future probabilistic model code on structural fire safety. The magnitude of the fire load is of particular interest as this variable has significant uncertainties. Fire load data are collected for different types of occupancies and the characteristic value of the variable is chosen to represent the 80 % fractile of the statistical distribution. However, it has been shown that the fire load varies in great extent within a specified occupancy type (e.g. book stores in shopping centres). Consequently, the validity of using 'generalised' fire load data in combination with 'local' variables (e.g. thermal properties and ventilation characteristics) when assessing the temperature-time curve must be questioned.

Fire load data, fuel type and fire compartment geometry are all variables that influences the outcome of the heat exposure model. Thermal properties of the structural element are considered to have great influence on the thermal action of a fire as they influence the net heat flow to the construction. The approach with several partial factors related to fire severity, danger of fire and active fire safety measures introduced in the Natural Fire Safety Concept[6] is promising and could form a basis for further development of the heat exposure model. However additional partial factors must be introduced related to the variability and uncertainty in fire load data and model uncertainties. It is also necessary to investigate at what phase in the heat exposure model that the partial factors should be applied. Adjusting only the fire load will most likely neglect some important aspects used to quantify fire severity. The next step in the development of a probability based design approach on structural fire safety will be to select those variables in the heat exposure model as well as the structural response model that has the strongest influence on the design of structures.

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