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Overview of research on fire effects in RC elements and assessment of RC structures after fire

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Subject review

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Overview of research on fire effects in RC elements and assessment of RC structures after fire

Procedures for estimating damage and remaining bearing capacity of RC structures after fire are presented in the paper. The damage to structural RC elements of a column, beam, slab, wall, and connection details, is presented in the paper. Although RC elements are generally considered to be fire resistant, their physical and mechanical characteristics are greatly influenced by temperature, while the bearing capacity before and after fire can be determined through a number of different methods that are presented in the paper. An overview of condition assessment for RC structures after fire is proposed, and methodology for taking further action is presented.

Key words:
fire, reinforced-concrete structures, test methods, condition assessment

Pregledni rad

Lucija Stepinac, Josip Galić, Hrvoje Vukić, Miljenko Haiman

Pregled istraživanja djelovanja požara u AB elementima i procjena stanja AB konstrukcije nakon požara

U radu su predstavljeni postupci procjene oštećenosti i preostale nosivosti AB konstrukcija nakon požara. Prikazana su oštećenja konstrukcijskih AB elemenata stupa, grede, ploče, zida te detalja spajanja. Iako se AB elementi općenito smatraju otpornima na požar, utjecaj temperature bitno utječe na njihove fizikalne i mehaničke karakteristike, a nosivost prije i nakon požara možemo odrediti primjenom više različitih metoda koje su prikazane u radu. Napravljen je osvrt na temu procjene stanja AB konstrukcija nakon požara te daljnja metodologija postupanja.

Ključne riječi:
požar, armiranobetonske konstrukcije, metode ispitivanja, procjena stanja

Übersichtsarbeit

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Übersicht der Forschung im Hinblick auf die Wirkung des Brandes in den Stahlbetonelementen und Einschätzung der Lage der Stahlbetonkonstruktion nach dem Brand

In der Arbeit wurden die Verfahren im Hinblick auf die Einschätzung der Beschädigung und im Hinblick auf die restliche Tragfähigkeit von Stahlbetonkonstruktionen nach dem Brand dargestellt. Es wurde auch eine Beschreibung der Brandwirkung auf die Konstruktionen dargestellt. In der Arbeit wurden auch die Beschädigungen von Stahlbetonkonstruktionselementen der Säulen, der Balken, der Platten, der Wand, sowie der Verbindungsdetails dargestellt. Obwohl die Stahlbetonelemente allgemein für brandwiderstandsfähig gehalten werden, beeinflusst die Temperatur wesentlich ihre physikalischen und mechanischen Eigenschaften, und die Tragfähigkeit vor dem Brand und nach dem Brand kann durch die Anwendung von mehreren verschiedenen Methoden festgelegt werden, welche in der Arbeit dargestellt wurden. Es wurde auch ein Rückblick auf das Thema der Einschätzung des Standes von Stahlbetonkonstruktionen gemacht, und es wurden weitere Verfahrensmethoden vorgestellt.

Schlüsselwörter:
Brand, Stahlbetonkonstruktionen, Prüfungsmethoden, Lageeinschätzung

1. Introduction

Fire is considered to be one of the main causes of damage to structures and loss of human lives. in 2015, as many as 2685 lives were lost to structural fires in the USA alone, while the related material damage amounted to US$ 10.3 billion [1]. In Europe, about five thousand fires are registered every day with four thousand fatalities annually, while as many as 70000 persons request hospitalisation due to direct or indirect effects of fires in households [2]. At the European level, structural fires account for material damage of €126 billion each year, which is 1 % of the total GDP. Fires can lead to infrastructure standstills, loss of data, fall in the value of shares, decrease in production, and increase in unemployment, and even to bankruptcy [3].

Figure 1 shows some significant fires in Europe in the time span of fifty past years, as related to the number of fire fatalities per 100,000 inhabitants. The size of the circle represents the location of the building affected by the fire, year in which the fire occurred, and the number of persons who lost life in the fire. In addition, significant fires with no fatalities that affected buildings belonging to cultural heritage are also presented (marked by temple icon on the map). Such accidents (such as Notre Dame, Figure 2) require significant efforts and enormous financial allocations for proper repair of the damage. The data are based on the information collected and analysed [4] although such information can be considered incomplete due to lack of documents on fire events in Europe. They can nevertheless serve as a good visual representation of the effects of fire events. The data for countries, i.e. an average number of annual fatalities per 100,000 of inhabitants, are based on the information provided by the International Association of Fire and Rescue Services for the 2013-2017 period [5]. As the information is lacking for some countries [5], the data for Portugal (2008-2010), Albania (2002-2004), Azerbaijan (2001-2007), and Northern Macedonia (2006-2010) have been taken from report [3].

Reinforced-concrete structures are very often perceived by laymen as being fully safe to the action of fire, especially when compared to structures made of other materials, such as steel and wood. Concrete is a non-flammable and fire-resistant material with a high effective diffusion coefficient, which slows down penetration of heat into the element exposed to fire. Despite that, RC structures can suffer considerable fire damage if exposed to elevated temperatures for a longer period of time, of may become fully damaged and unsuitable for future use. Elevated temperatures cause change in physical and chemical properties of steel and concrete, which can eventually result in the collapse of the entire structure, and potentially to the loss of human lives. The most recent example is the fire in the Grenfell Tower residential building in London where 72 persons lost life while 70 persons suffered injuries.

Physicochemical changes play a significant role in the load-bearing function of RC structures by greatly reducing mechanical properties of concrete. It is known that the first significant loss in strength occurs at the temperature of 300 °C. Elevated temperatures cause expansion of steel and hence loss in strength and elastic modulus. The change in properties of reinforcement

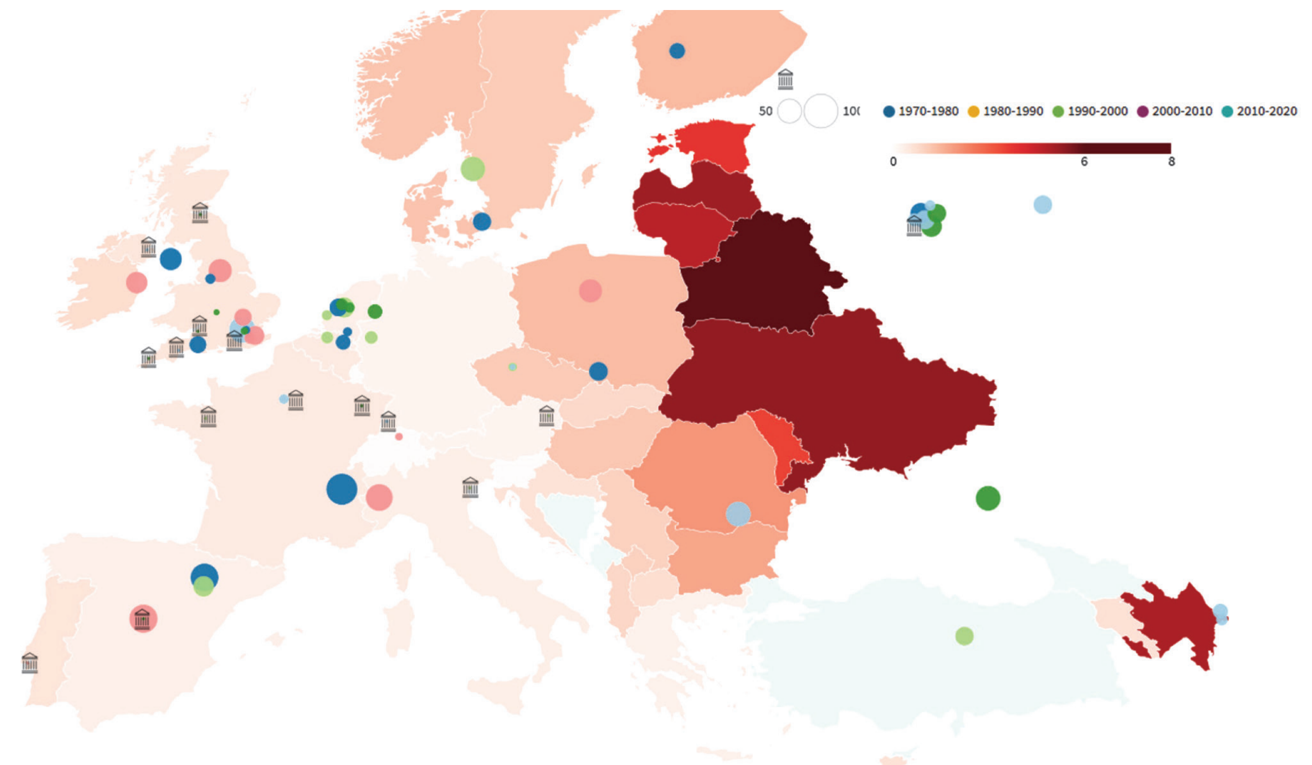

Figure 1. Map of significant fire events in Europe [4] as related to an average number of persons who lost life in structural fires [3, 5]

Figure 2. Notre Dame on fire [6], Grenfell Tower on fire [7]

exposed to elevated temperatures is of high significance to the structure. Basic processed occurring during increase of temperature in RC structures are presented in Figure 3.

Concrete heated to temperatures of over 500-600 °C becomes damaged to such an extent that it is no longer suitable for use in the structure [8]. That is why, the extent of degradation of external layer must be determined during testing [8]. An experimental testing aimed at detecting crack development processes and residual mechanical resistance of concrete after exposure to elevated temperatures was conducted by Xiang et al. [9]. Thirty-six samples were tested at temperatures of more than 600 °C by DIC (Digital Image Correlation) technology, and the following basic conclusions were reached: concretes of different w/c ratios have similar cracking and change in colour results if they are exposed to the same temperature. The fall in characteristic properties of concrete occurs after exposure to temperature of 400 °C or more. At the temperature of 200 °C the residual compressive strength was at 85 % compared to normal room temperature, at 400 °C the strength fell to 75 %, and at 600 °C it fell to 45 %. At the temperature of 600 °C the elastic modulus value fell for 80 %.

Each individual load bearing reinforced-concrete element is susceptible to a great number of influences when subjected to fire. The most significant are the duration of an increase in temperature, water content in concrete, type of aggregate used, and concrete composition. In addition to material characteristics, the behaviour of structure after exposure to fire is also influenced by the type of structural system because the force within a closed structure can cause irreversible damage due to influence of temperature [10]. Also, the fire fighting medium can greatly affect the structure, i.e. once the heated structure comes into contact with a great quantity of cold fire fighting substance (it is usually water), it suffers an additional thermal shock [11].

An increase in temperature of a RC structure is accompanied by physical and mechanical damage in structures as shown in Figure 3 where temperature is related to the risk of damage (higher temperature contributes to spalling, increase in crack size, damage to reinforcement, and even to collapse of the entire structure). The related risk is shown in Figure 4 where structures are classified according to temperature and vulnerability into categories from A to E, which is the way structures are evaluated for instance in the USA and Japan [43].

Being one of the most hazardous emergency situations, fire is included in standard documents. However, its action and the very analysis of fire activity are very often erroneously interpreted and defined. Standards and rules for the design of structures resistant to fire are constantly changing, and so new knowledge gained through scientific research and case studies should be continuously improved and implemented.

Fire resistance is the capacity of a structure, a part of a structure, or a structural element to fulfil normal functioning requirements (with regard to bearing capacity and/or fire propagation) for the defined level of load, defined fire exposure, and in the defined time period.

High level of resistance of reinforced-concrete structure to fire is the reason why cases of failure of RC structures are very rare. Although many fatalities have been registered in the case of fire affecting man-built structures, such death toll is very rarely caused by collapse of RC structures after exposure to

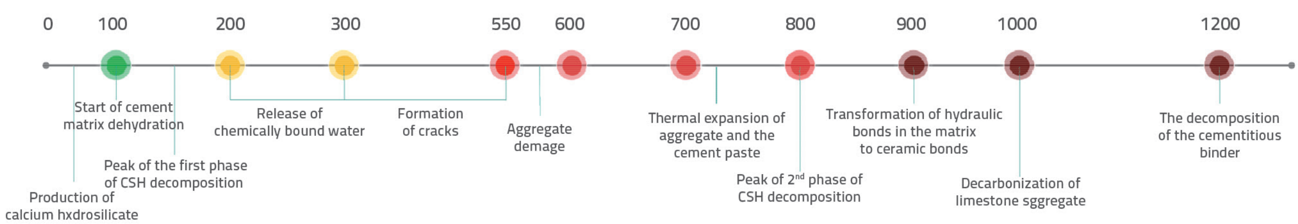

Figure 3. Basic processes occurring during increase of temperature in RC structures

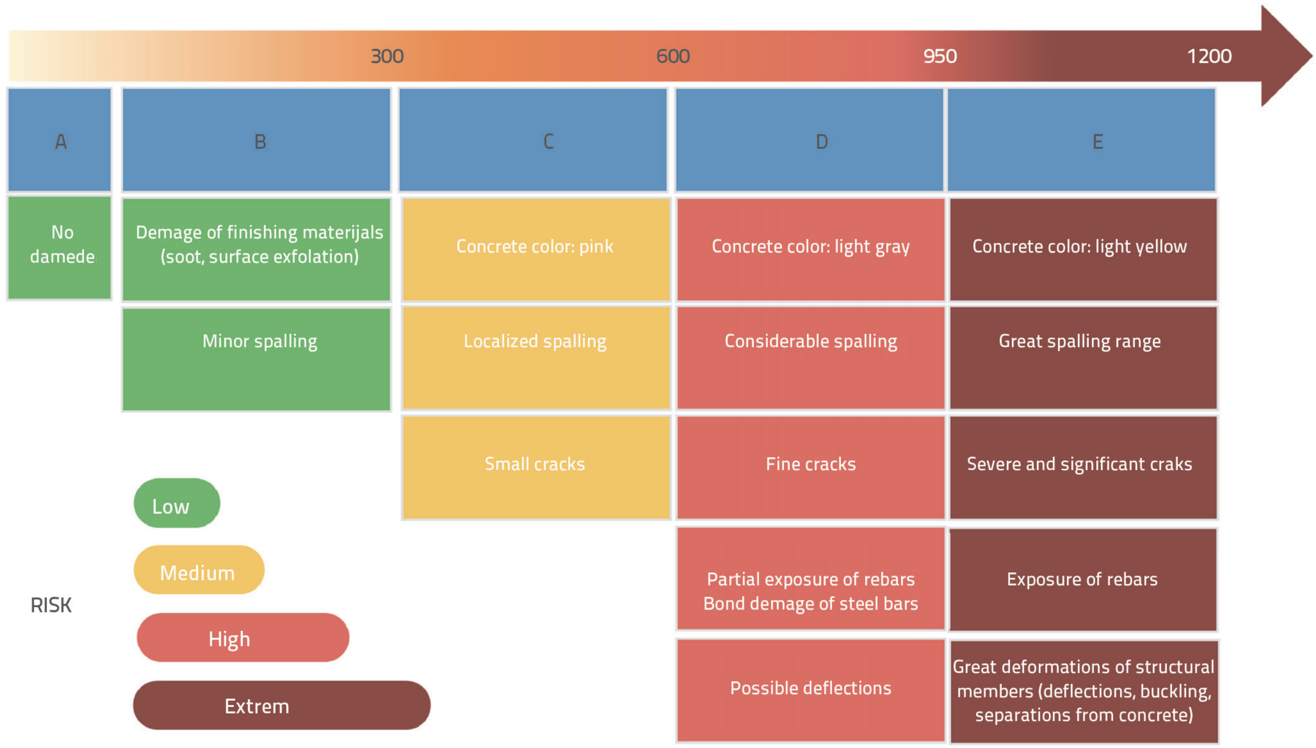

Figure 4. Level of risk to RC structures depending on increase in temperature

fire. Fire events are normally and almost inevitably followed by technical inspection of the structure and, after this inspection, by marking the elements that need to be replaced or, if possible, strengthened.

Extensive research has been conducted to estimate the remaining resistance of fire-damaged RC structures. The assessment of resistance of RC structures to fire action has focused on determining resistance to elevated temperatures using standard temperature-time curves, numerical studies and experimental analyses.

An overview of basic terms related to fires in RC structures is presented in this paper, the current testing activity relating to fires in RC structures is described, and a review of methods for assessing post-fire condition of RC structures is given.

2. Study of fire effect in reinforced-concrete elements

According to Eurocode [12], it is possible to use tabular data (EN 1992-1-2, Section 5), simple calculation methods for assessing individual elements (EN 1992-1-2, Section 4.2), and advanced calculation methods (EN 1992-1-2, Section 4.3).

The analysis of reinforced-concrete structures with regard to fire load involves selection of an appropriate cross-section and thickness of the protective layer. Load bearing elements of typical structures can be calculated using the so-called tabulated data [12]. In more demanding cases, fire resistance of structures can be determined by simple analyses in which the fire is regarded as an emergency event and it is on this basis that ultimate bearing capacity values can be checked [12, 13]. Tabulated data offer a recognisable calculation-based solution for fire exposure lasting no more than 240 minutes. If tabulated data according to EN 1992-1-2, Section 5, are used, then no further verifications are needed with regard to shear and torsion or verifications related to explosive spalling. Three main simplified methods are described in EN 1992-1-2, Section 4, and EN 1992-1-2, Annex B. The first one is called the "Method 500 °C isotherm". It can be applied for standard exposure to fire and it is valid for minimum width of cross section that is needed for fire resistance defined in EN 1992-1-2, Annex B, Table B1. The thickness of damaged concrete (a500) is regarded as being equal to an average isotherm depth of 500 °C in the compressive zone of cross section. It is assumed that concrete does not contribute to bearing capacity at temperatures in excess of 500 °C, while the remaining cross section of concrete retains its initial strength value and elasticity modulus. The second method called the Zone Method provides more reliable results compared to the first one, especially with regard to columns, and it is exclusively applied for standard temperature-time curve. It is based on obtaining a reduced cross section by ignoring the zone of damage at the side that is exposed to fire. The value is defined by reduction factors for each individual zone of cross section. The third method, the so called "Method based on estimation of curvature" (EN 1992-1-1, Section 5) enables assessing condition of reinforced-concrete cross section exposed to bending moment and longitudinal force. It can be applied for columns with pronounced second order effects.

Advanced calculation methods include the following three models (EN 1992-1-2, Section 4.3). The first one is the so called thermal response model (based on the theory of the transfer of heat and thermal actions presented in EN 1991-1-2). International temperature-time fire curves (the best known one being the Standard ISO 834 curve) can be used but only if properties of materials (concrete and steel) are known for every relevant temperature stage. The mechanical response model introduces into calculation the change of mechanical properties due to change in temperature. The effects of stresses and strains thermally generated due to an increase in temperature and temperature differences must be taken into account. The compatibility requirement in the form of allowed deformation at structural level must also be met, where nonlinear influences of geometry and boundary conditions should be taken as relevant.

2.1. Column

Fire resistance of a concrete column for a standard fire can be determined by a number of methods. The simplest determination would be via tabulated values from EN 1992-1-2 [12]. However, the use of tables is limited to immovable columns up to 6 m in height. The second type of determination of fire resistance of concrete columns is via the so called "advanced calculation methods". An advantage of these methods is that, unlike tabulation calculation methods, they are not limited to a specific type of element. Simplified calculations have been developed for rapid verification of fire resistance. Their greatest deficiency is the impossibility of observing the structure as a whole, i.e. the limitations are given only for an individual element regarded for the case of load by standard fire.

Most standards, including ACI 216 [14] and EN 1992-1-2 [12], contain provisions for determining fire class for load bearing elements such as columns. The existing standard for the calculation of fire resistance of concrete elements (EN1992-1-2) sets the load bearing value for fire load in the case of slender columns (for the requirement: R30). Fire resistance is influenced by the following parameters: column slenderness, cross-sectional dimensions, length, boundary conditions, and load eccentricity [15]. Properties of reinforcing steel are also highly significant [16]. The percentage of explosive spalling, i.e. loss of cross section during fire, can also be influenced by concrete moisture level (Figure 5). Additional parameters are the selection of aggregate in concrete, protective layer thickness, etc. A greater area of longitudinal reinforcement in the concrete column cross-section can also contributed to fire resistance.

In their research, Buch et al. [15] conclude that an increase in degraded concrete surface from 15 % to 80 % leads to the fall in bearing capacity by 77 %, and that it is precisely the new empirical expressions that should take into account explosive spalling of concrete as a significant parameter for the calculation of fire resistance of concrete elements. A sudden fall in bearing capacity of high-strength concrete will occur due to local buckling of longitudinal reinforcement in column. The fire resistance of RC columns will increase by providing for proper longitudinal and transverse reinforcement details, i.e. by reducing the distance between stirrups.

Gernay [1] studied fire resistance of RC columns. Fire resistance is defined as the capacity of a structural element to survive exposure to fire, including the structure cooling time. Numerical method involving nonlinear finite elements was used to analyse a column in the heating and cooling phases in order to gain information about fire resistance. The final objective was to obtain a simplified method for calculating fire resistance of RC columns. By placing focus on concrete columns, this paper covered the base of 74 standard fire resistance tests. The comparison of numerical model results and test results revealed that modelling can be used to predict RC column failure in standard fire. In the second step, 74 columns were tested in fire that also involved the cooling phase. A special attention was paid to proper definition of the influence of the cooling of material, including subsequent loss of compressive strength of concrete, irreversible loss of concrete properties due to contact with moisture, and explicit evaluation of the propagation of creep.

Figure 5. Column damage by fire: a) column blackening; b) cracking; c) exposed reinforcement

An iterative calculation procedure was conducted separately for each individual column by extending the time of natural fire so as to obtain the shortest fire that would lead to full combustion of every individual column, thus defining the DHP phase (temperature increase phase). The DHP phase must be shorter that the time needed to achieve fire resistance (R). In other words, the difference between the DHP and R increases with R. This difference is a good indicator of the susceptibility toward a delayed failure of structural elements. That is why a close connection was established – for practical purposes – between the fire resistance (RA) and resistance defined by the DHP phase. It is recommended to rely on DHP and R values for classification of structural elements, rather than on fire resistance (R) only [1].

In their column testing under real fire, Lo Monte et al. [17] concluded that it would also be highly significant to define the cooling phase during the real fire modelling activity.

The resistance to longitudinal force and serviceability limit state of RC columns were considered in [18]. RC columns were strengthened with a hollow steel pipe. The following main conclusions were reached: the failure of RC column strengthened with hollow thin-walled steel pipes is manifested through local buckling, while the RC column itself fails in shear; the strengthening restored the bearing capacity of damaged column to the level existing before the fire load, while the resistance to longitudinal force was even exceeded as compared to the situation prior to fire, and the level of ductility was even increased.

2.2. Beam

In the research published by Yang et al. [19], the main objective was to quantify the influence of various parameters on the behaviour of a reinforced-concrete T-beam confined by prestressed steel straps (PSS). Twelve samples of fire-damaged T-beans strengthened against shear by PSS method were considered. The following conclusions were reached by analysis of experimental results: during the fire testing lasting 60, 90 and 120 minutes the temperatures of 900 °C, 990 °C and 1025 °C were reached, which compares quite well with the temperature-time curve for standard fire according to ISO 834. However, an increase in temperature within an element is not easy to detect. The differences in the change of temperature between the bottom part and the flange of a T-beam are relatively high, amounting to 290 °C, 400 °C and 580 °C. The damage to concrete beams increased with an increase in the time of exposure to fire (Figure 6), which was manifested in the reduction of stiffness and shear capacity. After the T-beam strengthening by PSS method, the shear resistance and stiffness of RC beams exposed to fire was restored, and steel cables contributed to beam ductility, while also preventing development of cracks. The reduction of spacing between steel cables led to increase in the bearing capacity and stiffness of beams (it is important to note here that an excessive prestressing could limit shear resistance) [19].

In their paper [20] Elshorbagy et al. presented a numerical parametric fire-related analyses using Ansys software in order to obtain the following parameters: compressive strength of concrete, protective layer, and lateral stiffness of the beam. The algorithm for calculation model is based on nonlinear analysis of finite elements, and it combines several fire scenarios in order to obtain structural response to fire action. The validity of the calculation model was determined through experimental testing. The proposal is to simulate the explosive spalling phenomenon, which can be conducted by hydro-thermal analysis, as this approach also takes into account the influence of water on the degradation of concrete cross-section as related to explosive spalling, which is considered to be the main cause of this phenomenon.

During exposure of RC elements to fire, thermal gradients are created as the temperature advances from the surface toward the interior of the cross section. The existing methods are: equal area method (thermal), maximum temperature method (thermal), energy method (thermal), load capacity concept (mechanical), and maximum deflection method (mechanical). In order to facilitate the calculation, time equivalent methods are needed for estimating the strength of normal fire on the basis of duration of standard fire [21]. Using such time equivalent methods, it is very easy to obtain real fire load based on available data, testing, and computer programs that are based on the standard fire curve. Currently available methods of this type are inaccurate in assuming thermal gradient of RC element exposed to fire load. Kuehnen et al. [21] developed the AITP method (AITP = average internal temperature profile) which has proven to be more accurate in the interpretation of thermal gradient. The authors claim that, based on the AITP method, designers can rapidly relate the severity of real fire with that of standard fire, so as to obtain a satisfactory behaviour of structure when subjected to fire.

In their work, Albuquerque et al. [22] made appropriate comparison on a RC beam subjected to bending with various boundary conditions (articulated and fixed). It was concluded that the resistance to fire action greatly influences boundary conditions in the beam and so, in some cases, this resulted in a 100 % increase in bearing capacity. A smaller quantity of visible cracks was observed in the beam with supports that assume displacements only (without rotation). A significant lateral deformation was observed in the case of fixed supports (where both displacement and rotation are prevented), and this deformation resulted in failure of longitudinal reinforcement, which in some cases affected both top and bottom zones.

The following facts must be taken into account in the case of prestressed beams: cold-finished prestressing steel is more sensitive to fire compared to hot-rolled prestressing steel, reduction of cross-section leads to lower protection of prestressed steel, prestressed elements designed before introduction of standards do not have a protective layer that would ensure steel protection in the case of fire, poorly realized connections between bearing elements do not enable redistribution of internal forces caused by fire load, which directly influences the loss of global stability of the structure. In paper [23] the authors test a simply supported beam (pretensioned and posttensioned) exposed to normal fire in order to enable determination of possibilities of delayed failure of structures (in the cooling phase) and to explain the role of individual parameters including: duration of fire, fire force, and cooling rate. In both cases, pretensioned and posttensioned concrete elements reveal continued heating of prestressed steel even after the ultimate cooling point has been achieved, which may lead to delayed failure of structural

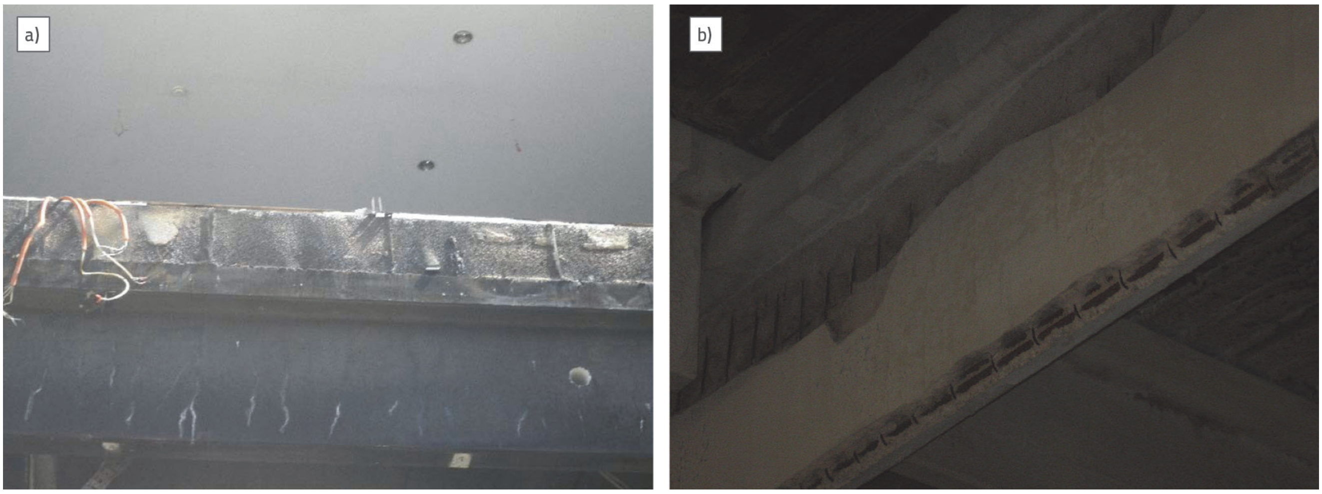

Figure 6. Beam damage caused by fire: a) visible cracks at external face of the beam; b) reduction of cross-section, exposed reinforcement

elements, especially in brief fires. A longer cooling phase causes greater temperatures in prestressed steel, especially in the steel with a more massive cross section. The remaining deformations are significantly reduced by doubling the cooling rate from 5 to 10 °C/min.

In recent times, research in this area has greatly focused on the categorisation of the influence of fire on reinforced-concrete and prestressed concrete elements at the cooling phase. The investigations include experimental research on posttensioned concrete slabs previously exposed to a localized fire [24], calculation of reinforced-concrete columns [25, 26], reinforced-concrete frames [27], prestressed concrete beams [23], and studies that define the remaining coefficient of bearing capacity of concrete beams after a fire event [28]. Numerical analyses can be used to anticipate failure of structures during and after cooling phase of various structural elements and materials [29].

2.3. Slab

Fire-provoked shear failure of standard structural elements occurs quite rarely but, in case of flat slabs, the situation is different. Generally, the following should be taken into account in every analysis of fire resistance of flat slabs: accurate fire scenario, thermal analysis, and structural behaviour. The real fire scenario was determined in [30] by testing fire load on an actual size model. An example of an underground carpark is shown in [23] to show that an increase in temperature in the slab-column structural system leads to an increase in transverse force at the connection with the column, which in turn leads to an increase in longitudinal force that is 50 % greater compared to room temperature. In comparison with standard fire given in ISO 384 [31], it was proven that the standard fire curve shows greater deterioration of material characteristics as related to real fire while, on the other side, the redistribution of internal forces is underestimated.

Although the column connection across the flat slab is considered a critical zone that can lead to collapse in a fire scenario, only several tests have so far been conducted about this significant issue. It has been concluded that, in the case of flat slabs and columns, an increase in temperature directly influences material characteristics and causes dramatic decrease in mechanical and thermal properties. Thus even a 100 % decrease is possible at the temperature of 1000 °C. In addition to the decrease/loss of the above mentioned properties, there is also a drastic increase in internal forces as related to the condition prevailing at room temperature. The quality of concrete and percentage of longitudinal reinforcement have shown an unfavourable influence or greater level of damage in fire, while the influence of shear reinforcement was shown to be minimum. Furthermore, this test has revealed a difference in mechanical properties of elements that were exposed to either sudden or gradual cooling. A greater level of damage has been observed at sudden cooling compared to gradual cooling [30].

Knowledge of material strength and loss of prestressing force is of key significance in the estimation of bending strength of slender prestressed concrete slabs. Much research still needs to be conducted to gain better understanding of the relationship between the thermal expansion and fall in stress level in cables [32]. In their paper, Musmar et al. [32] conducted a numerical analysis using the finite element method to study a slab exposed fire in order to predict the ultimate point of structural failure. A 60-minute fire affecting a prestressed concrete slab resting on concrete columns was considered. It was finally concluded that it would be better to pull down and rebuild the structure than to implement measures such as: full replacement of the prestressed slab, and/or strengthening of columns at levels below and above the slab.

An accurate prediction of the behaviour of structural elements under fire exposure conditions is possible only in cases when all parameters for numerical analysis are know. If parameters such as deformations are neglected, the error of results can increase by more than 20 %. The analysis that takes into account the condition of structure after the cooling phase is considered relevant for prediction of the remaining bearing capacity of RC elements after exposure to fire load. However, even this analysis has to be extended by additional experimental testing. It is indispensable to conduct experimental and numerical analyses for the calculation of real resistance of concrete elements under fire exposure conditions [33].

2.4. Wall

Reinforced-concrete walls are often used as vertical barriers in structures because of their relatively low cost, good aesthetics, better use of space, and simplicity of construction. Compared to masonry walls, they offer much greater bearing capacity both in and out of the plane. The fire in RC walls does not act uniformly along the height of cross-section, and thermal gradient may cause uneven degradation in the cross-section of the wall, which leads to eccentricity of the force of gravity, i.e. the fire load on the wall can lead to the loss of out-of-plane stability [34].
Kumer et al. conducted a numerical analysis on a 3D model using the Ansys program package, and verified the analysis by experiment. A special attention was paid to the introduction of temperature-related lateral buckling in the analysis of behaviour of RC walls under fire conditions. The following conclusions were made: A high thermal increment leads to creation of steep thermal gradients within the RC wall, which in turn leads to out-of-plane instability of the wall. The response of the structure affected by fire passes through three phases: the first phase involves a rapid increase of the out-of-plane displacement, the second phase is characterised by displacement of neutral axis, which is due to degradation of material at elevated temperature, while the third phase is marked by domination of displacement in neutral axis due to thermal gradients. Results obtained by 3D model can be used for developing a simplified 2D numerical model by which the fire-resistance of load bearing RC walls could be calculated.

2.5. Frame

A proposal of numerical calculation for estimating global resistance to fire load in reinforced-concrete 3D frames is presented by Magisano, D. et al. [35]. A simple, accurate, and efficient numerical procedure for calculating contribution of longitudinal force and biaxial bending of RC frame under fire is derived in the paper. It is based on the Minkowski sum of ellipsoids, where each ellipsoid represents a contribution of the subdomain of cross-section to the total area. Thus obtained conditions are time-dependent. These criteria can easily be used for estimating safety of structures, by simply verifying local bearing capacity of cross-section. However, an excessive length of structure and ductility due to confinement and increase in temperature, enable redistribution of stress over the frame, which makes verification of cross-section extremely conservative. For these reasons, a new global analysis of fire has been proposed. This type of calculation provides an insight into duration of fire load and redistribution of stress, and into the possibility of detecting time limitations, i.e. the time exposure to fire that leads to failure of the structure.
The influence of fire on the behaviour of a multi-storey concrete structure is presented in the paper by Elbayomy, M. et al. [36]. The testing was conducted using the AEM (Applied Element Method). Deformations and stresses in elements exposed to elevated temperatures were considered. The following conclusions were made: maximum temperature in fire has a significant influence on the behaviour of various types of structural elements, compressive strength of concrete has a significant influence on the deformations in boundary areas of slabs and beams, the column reinforcement percentage exerts a considerable influence on the deflection of boundary slabs and on the deflection of boundary beams. It can therefore be concluded that the duration of fire and temperature are the most significant parameters for calculating RC columns with regard to fire load. The temperature of 900 °C can lead to progressive failure of individual structural elements, or even of the entire structure. In numerical analyses, properties of material (its deterioration) during elevated temperatures are indispensable for the calculation model. The location of fire is also a significant factor in the design process as, for instance, the fire in corner parts of the structure is more critical that the fire that occurs in the central area and, similarly, the fires at lower levels are more harmful than the fires at higher levels.

2.6. Details

Various reinforcement details for the beam-column connection within a RC frame are considered for the situation prior to and after the fire load [37, 38]. Numerous research activities have focused on obtaining simplified models so as to idealise behaviour within the beam-column connection during the fire load. Stylianidis et al. [39] presented a detailed study on the mechanism of progressive failure and gave a proposal for simple realisation of connection details. A special attention was placed on the comparison of experimental results and theoretical predictions of interaction between the longitudinal force and bending moment for the room temperature conditions prior to and after the fire (Figure 7). It was established that comparison between experimental testing and simplified analytical models can be made using expressions for the case prior to and after the fire. Numerical calculation offers a reliable and efficient approach to the detection of key phases, including the initial stiffness and bearing capacity, and the limit state of deformation. The proposed calculation model takes into account material characteristics and adherence after fire and the remaining area of reinforcement plasticisation.

Figure 7 Damaged details at the beam – column connection

2.7. Bridge

The current literature shows that there is a lack of published papers about the standardisation of criteria on the inspection, assessment and repair of fire-affected bridges. In most research conducted so far, fire resistance is determined through comparison of theoretical results based on numerical models and experimental research. However, real structural performance is worse that the theoretically assumed performance, which point to the probability of errors in the estimation of bridge condition [40].

3. Assessment of post-fire residual strength of RC elements

It is important to determine the current structural condition before the start of repair and renovation of structures damaged in fire. Presently available information and technologies enable better resolution of structural fire issues, as well as better education about procedures to be taken before and after a fire event [41]. Non-destructive testing methods (NDT) have recently been increasingly used for assessing condition of reinforced-concrete structures.

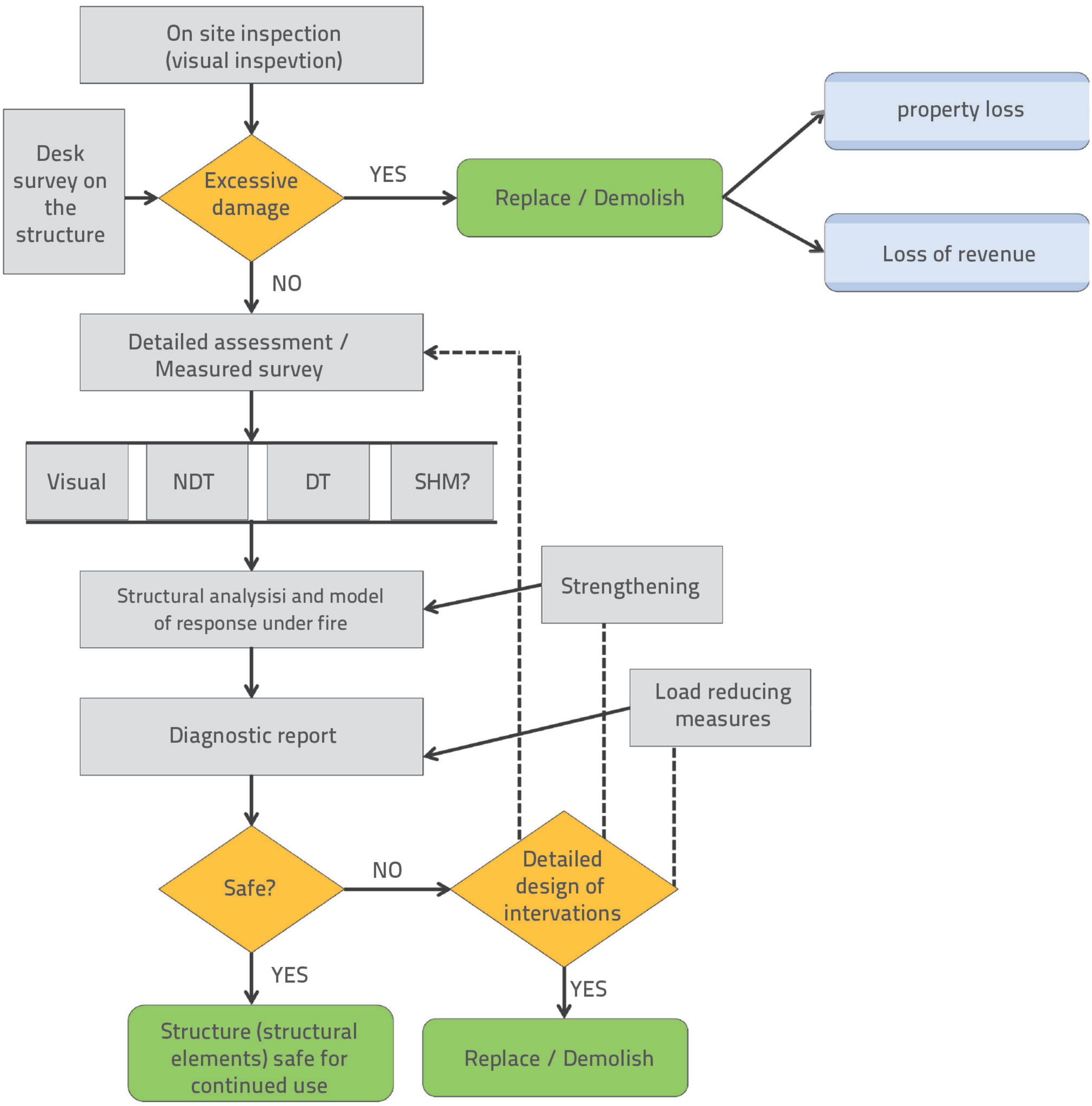

Figure 8. Assessment of condition of RC structures

Concrete structures are susceptible to damage due to service load, fatigue, adverse ambient conditions, and extreme occurrences. All these effects can harm the integrity of concrete structures, which is manifested through damage to reinforcement of concrete itself.

A reliable and rapid assessment of post-fire resistance of reinforced-concrete structures is indispensable for detecting the level of structural damage prior to intervention of fire-fighting and rescue units. An economic assessment of damage is of crucial significance when making decisions on either repair or full removal of the structure. Taking into account the high price of replacement of load-bearing elements, necessary equipment, and also the cost of the damage assessment study, it becomes clear that a simple and rapid testing procedure is indispensable [42]. In the case when repair of damaged structure is not possible, or in the case when such repair is possible, preliminary information about the level of damage caused by fire should be sufficient for the repair or removal of structure without expensive and sophisticated assessments.

When discussing condition assessment, we are talking about establishing relationship between the fire as such and properties of structural elements. This by itself is not an easy task as any error can directly jeopardise human lives. Procedures for assessing condition of concrete structures can easily be explained through flow chart presented in Figure 8.

Buildings affected by elevated temperatures during fire events exhibit various behavioural patterns of premature ageing or deterioration. Visual inspection is the first step in structural condition assessments. Although visual inspection is the foundation of condition assessments, and also the first (and sometimes the last) step, it is very often impossible to accurately determine the remaining bearing capacity of load bearing elements affected by fire by this method alone. Although surface damage due to loss in cross section can in fact be observed, mechanical characteristics or redistribution of stress within elements can not be defined by visual inspection only. It is difficult to use this method to accurately estimate the temperature as the assessment itself depends on the experience of the person making the assessment [43]. Other methods that can be used in temperature assessment include non-destructive methods such as the ultraviolet spectrum method [44], ultrasound spectroscopy method [45], X-ray diffraction method, and petrographic analysis [46], as well as the method for measuring quantities of repeated absorption of carbon dioxide. The results may depend on the method selected, but also on chemical composition of concrete.

After initial visual inspection and review of literature about the structure itself (desk survey), material and structural uncertainties that cannot be seen by naked eye must also be solved. In parallel with or immediately after a detailed visual inspection of structure, numerous characteristics can also be determined via non-destructive, semi-destructive, and destructive methods. The data obtained in this way are used in structural analysis and in assessment of the remaining bearing capacity of the structure; if it is insufficient and if there are certain uncertainties, then detailed testing can be repeated or the existing load can be reduced, or the structure can be strengthened or, finally, it can be pulled down if none of the other measures seem cost-effective. The remaining bearing capacity also depends on concrete strength. Tolić et al. [47] show, through results obtained by testing lightweight concrete, that the cooling phase of concrete elements is highly significant. The remaining bearing capacity exhibits a fall for additional 20 % after 96 hours as related to the condition prevailing immediately after cooling. Basic post-fire condition assessment procedures for RC structures are presenetd in Figure 9.

The information about cracking can assist in timely and proactive management of concrete structures. Recording sensors can be used in this respect, while visual inspection is considered ineffective both with respect to cost and time. Crack detection algorithm can be used to conduct quantitative analysis of strength or crack length, so as to estimate the level of safety [48]. Despite the use of sensors, reliable detection of cracks on damaged surfaces is always a challenge.

Cioni et al. [49] used thermomechanical and microstructural analyses as well as ultrasound measurements to assess post-fire damage to reinforced-concrete structures. The microstructure of concrete changes with an increase in temperature, which negatively affect its strength. Only one non-destructive method is not considered sufficient for assessing damage to concrete elements. Therefore, several methods should be used in combination, such as the sclerometer (impact hammer), ultrasound, microstructural analysis, and interpolation of temperature exposure. Eighteen reinforced-concrete columns, beams and cylindrical samples were tested in the paper. Test results, based on applying equal load to cylinders and to RC beams and columns, were quite consistent. All samples were subjected to non-destructive testing prior to and after the fire. The strength of each individual sample was tested. The correlation between NDT methods and the remaining strength obtained by testing at various fire load durations was analysed.

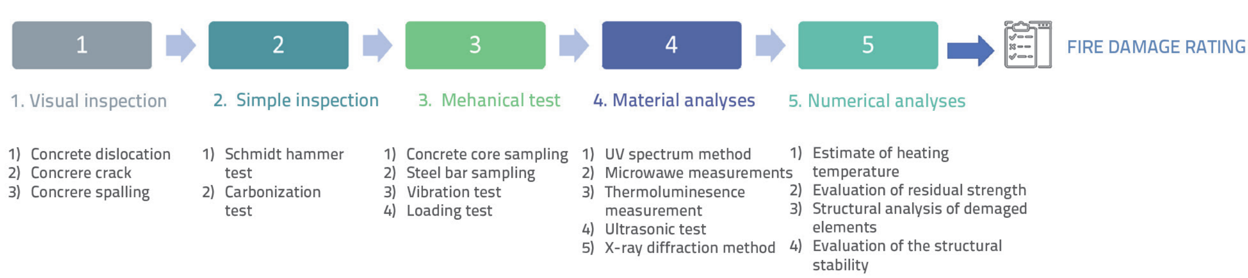

Figure 9. Basic post-fire condition assessment procedures for RC structures

The following main conclusions were reached: compressive strength values obtained by sclerometer can not be compared with results obtained by testing samples under compression, because sclerometer (impact hammer) estimates the strength value based on surface strength. The results obtained by ultrasound and iterative methods have proven to be a useful tool for estimating mechanical strength of concrete elements exposed to fire, and the microstructural analysis has also proven to be good for assessing damage to concrete.

The methodology for assessing the remaining bearing capacity of concrete structures after fire load is presented in [50]. The methodology itself is based on real example of fire action in a residential building with the focus on the last span of a continuous concrete slab affected by fire. The result is a reliable estimate of maximum allowable load to be imposed on slabs. The proposed methodology is useful for making decisions about the possibilities of using buildings after exposure to fire. The space and time distribution of temperature in structures during an actual fire load is mostly unknown. That is why designers must use indirect methods to make realistic estimates of the severity of fire events. The remaining bearing capacity of elements affected by fire can be assessed by numerical models and through calculations, but it is inevitable to simplify assumptions and model uncertainties. It is sometimes difficult to assume structural characteristics because of the lack/absence of design documents or because of degradation or modifications that might have been made during the life of the building, i.e. before the actual fire load. A reliability calculation for concrete structures is presented in the paper, but the same methodology can also be applied for other types of structures.

This approach links the philosophy of reliability of calculated bearing capacity in the period prior to a fire event to the time after the fire event. The methodology of reliability includes the following steps: observation of fire load, collection of data, and several steps for estimating maximum allowed load that can be imposed on structures. It would be appropriate to conduct a forensic investigation (on-site measurements, interviews with fire-fighters and designer of the structure), and to carry out simplified methods, advanced numerical modelling (FEM), reliability calculation, and engineering assessment. The data collected in this way would enable application of the concept of reliability in the assessment of structures after fire events, which is in line with the Eurocode (EN 1990) philosophy. The methodology was tested on a real example of a fire-affected concrete slab (the end span of that slab was tested). The following conclusions were made: a) lack of information greatly complicates assessment of the condition of structures after fire, and b) iterative method takes into account testing in the fire zone, numerical model (comparison of deformation data with measurements on the structure itself), and expert evaluation by a competent person.

A repeated assessment [51] can be made in order to obtain maximum allowable load after fire action, including the previously defined safety factor that takes into account the damage caused by fire (and after fire). This method is based on reliability concepts and hence enables explicit quantification of the remaining level of safety. Rapid calculation is based on simplified analytical expressions and on the assessment interaction diagram (AID). The aim of this method is to make decision on future use of the structure taking into account all above mentioned parameters. It is important to note that by using the reliability concepts we do not take into account the ultimate state of serviceability (deformations), but only the ultimate bearing capacity.

A currently popular trend involves development of probabilistic approach for assessing fire-related losses incurred by the structure itself or by its elements. Using this approach, designers can estimate the price of repair or replacement of the structure. Such an approach also enables calculation of additional protection aimed at reducing final costs, in the event of an earthquake action. The existing fire-load design methods are general and involve assessment about whether the structure will fail or will not fail. It is necessary to meet the prescribed criteria about the time of resistance to fire, which are almost completely based on the preservation of human lives. The condition assessment is conducted according to ISO-834 [31] which represents only one among many possible fire events that could occur.

Standard fire testing, which served as basis for drafting general guidelines, is limited to the verification of one structural element, rather than the structure taken as a whole. Quantification of the effect of fire imposed on structure is a complicated procedure. During the design or assessment of condition of a fire-affected structure, it is difficult to define the real time-temperature relationship. Visual inspection can provide basic information about the effects of maximum temperature on structural elements, such as the change in colour, or similar aspects. Various scenarios that could be applied in the calculation of structural response can be assumed through fire intensity measurements. Such an approach could enable quantification of the scale of damage, and better understanding of the damage to structural elements or to the entire structure.

The assessment of fire intensity through sensitivity curves enables prediction of the state of damage of, for instance, a column, as was done in the research made by Rush et al. [42]. In order to define intensity of a real fire on the time-temperature curve for three reference heights, three column levels were converted into areas below the standard ISO fire curve: total area under the curve; total area under the curve but above 150 °C; and total area under the curve but above 400 °C. These fires were applied for measuring intensity using sensitivity curves which show the probability of column being affected by one of the phases of damage. The probability for Level 4 damage (ds4) for the fire intensity in the 60th minute is about 40 %; the probability for Level 3 damage (ds2) is about 80 %; the probability for Level 2 damage (ds2) is about 97 % and the probability is almost 100 % for Level 1 damage (ds1). By limiting the curve to values below 400 °C it can be expected tha the column will be in phase 0, i.e. that it will not be affected by the fire.

Naser et al. [52] combined the modern concept of using computer intelligence for the estimation of damage caused by fire, including also resistance of concrete columns to fire. Computer intelligence, i.e. genetic programming, enables a highly accurate determination of damage to concrete elements, as well as detection of fire-resistance of concrete columns for fire exposure lasting more than 4 hours.

The fire damage estimation system developed in study [53] takes into account the database obtained by inspecting the following variables of RC structures: change in colour (CD), crack width (CR), degraded concrete depth (SP), heating temperature (TP), ratio of designed to measured compressive strength of concrete (RCS), and concrete carbonisation depth (CA). The following conclusions were made: currently available methods for estimating fire-related structural damage greatly rely on the experience and knowledge of the person doing the inspection of the structure, which is why it is very difficult to obtain unbiased and reliable results. However, the fire damage diagnosis system (FDDS) proposed in this study can offer consistent and objective results, because the comprehensive analysis of inspection and on site testing results can be made based on fuzzy theory. The application of the FDDS method is recommended in this area because of good correspondence with the results obtained by professional inspection using the AIJ method. The FDDS method can contribute to more reliable assessment of the level of fire damage of RC elements. The problem with this method is that the fire damage can only be assessed for an individual element, i.e. it can not be applied for assessing condition of the overall RC structure. Additional investigations are therefore needed in the future so as enable full application of the FDDS method.

4. Conclusion

The most often used methods for the repair of RC elements subjected to bending action is the cross section increase method, strengthening by fibre reinforced polymers (FRP), and strengthening by steel plates (connected with concrete by bolts) [54-56]. The cross section increase method is simple, sophisticated, and widely applicable. It can be used on load bearing concrete elements of beams, columns, slabs, foundations, etc. It is also known as the most cost-efficient method, but it necessitates reduction of the structure's internal clear span. The FRP strengthening method is applied by connecting FRP (bars, laminas, strips) to the external walls (by gluing or anchoring) or by inserting such FRP elements into the previously cut grooves. It is also a widely applied methods due to its advantages such as the low unit weight, high strength, and resistance to corrosion [50]. Strengthening by wrapping and anchoring steel plates onto the walls of concrete elements increases the tensile and compressive bearing capacity of RC beams (while wrapping by FRP increases the compressive strength only). Steel plates are anchored into concrete by means of anchor bolts. This type of strengthening also results in an increase in shear strength, stiffness and ductility of concrete elements.

Just like any other extraordinary action, fire must be taken seriously from the first phase of design, to the realization stage, and all the way to the maintenance of structures. Although it has been generally acknowledged that wooden or steel structures are greatly affected by fire (which is sometimes not the case), laymen often consider that reinforced concrete structures do not burn and, thus, that they can be very little affected by a fire event. From the engineering point of view, the design of RC structures for fire action is quite unambiguous, although there are still many uncertainties that have to be taken into account. A number of physical and chemical processes take place in RC elements at elevated temperature. These processes, usually not discernible by naked eye, contribute to the reduction of bearing capacity of the affected elements. According to recent research, the cooling phase is an especially hazardous period within the fire process, i.e. this phase is in many ways even more hazardous, compared to other structural materials. As already mentioned, proper design, construction and maintenance of structures contribute to proper fire resistance of structures. Nevertheless, we have to acknowledge that fires still occur, and will continue to occur in the future, which is why the experience gained through assessment of condition of structures after extraordinary events, such as earthquakes or fires, must be wisely used to continuously improve standards and rules relating to the behaviour of all types of structures. Assessing condition of the existing structures is a demanding process in itself, regardless of whether the remaining mechanical resistance of structures not affected by extraordinary events is assessed, or if assessment is made after an extraordinary event [57]. The post-fire assessment of RC structures is greatly dependent on the experience of engineers and authorized/certified persons, which is why a continuous education of experts is an indispensable precondition for successful assessment.

A general principle emphasized in leading international structural design standards is the protection of human lives, i.e. in the case of fire, it is the design time needed to evacuate people without any fatalities. Fire related design is first of all dependent on the condition of the material and structure [58], and on the interpretation of the certified engineer relating to the remaining bearing capacity and level of damage of the existing RC elements. In consultation with certified engineers, the decision makers, owners and/or investors make decision about whether it would be cost-effective to either "save" or completely remove a particular building.

This paper provides a brief overview on the effects of fire on RC structures, while also focusing on the influence of elevated temperatures on properties of RC structures. A brief review of regulatory framework is given, and it is shown that fire load is much more that mere consultation of tabulated data, and that many possible fire action scenarios exist for each individual structure. Standards and rules for the design of fire resistant structures are constantly changing, and so new knowledge acquired through scientific research and case studies must continuously be improved and implemented. As specified in Eurocode [12], use can be made of tabulated data (EN 1992-1-2, Section 5), simple calculation methods for the assessment of individual elements (EN 1992-1-2, Section 4.2), and advanced calculation methods (EN 1992-1-2, Section 4.3). Methods for estimating the remaining mechanical resistance and stability of RC structures after fire action are considered in the central part of the paper. Basic procedures and related problems are described in detail, and a flow diagram for estimating condition of RC elements after fire events is recommended. Necessary procedures and investigations, forming an integral part of reports on the assessment of mechanical resistance and stability of RC structures after fire events, are also presented.

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