Structural integrity of buildings under exceptional fire

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ABSTRACT: The paper is summarizing the European knowledge in the field of the behaviour of the structures made of the main structural materials under fire action. The new findings are stressed to prepare background for studying of the exceptional fire conditions. The exceptional fire is the commonly observed problem of the fire acting after or together with other accidental loading as earthquakes, windstorms, heavy snow loading, gas explosions, bomb blasts, impact of vehicles, rapidly changing loading situations (temperature) or ice loads. The exceptional cases include also the situations not taken account during the design or in fire the unexceptional conditions due to the different fire scenario and burning materials (different to expected cellulose or hydrocarbon fires). The structural integrity is the major goal of the designers to ensure the resistance of the structure under accidental actions and enlarge the safety under exceptional condition. It is guaranteed by the robust of structural elements and its joints, which is described by its balanced stiffness, strength and ductility between its members, connections and supports.

1. INTRODUCTION
The existing European findings, models, and regulations about the structural safety are generally, with few exceptions, concerned with rules aimed to ensure an adequate safety level of constructions under, normal loading conditions. The overall structural safety is investigated simply by assuming an increase of the multipliers of the service loads up to reach the collapse value. This approach has led to a satisfying degree of accuracy in the prediction of the safety margins under serviceability load conditions. A greater accuracy in the evaluation of structural safety is possible when a probabilistic or semi-probabilistic approach is followed in the determination of both actions and structural resistance. In this way is possible to get a good investigation of structures subjected to random actions. These approaches form the basis of most recent developments in the field of regulations and are now part of any relevant structural code, including of course the Eurocodes, where specific allowance for accidental loading conditions is made.

Fire resistance is used to characterize the performance of elements of structure in fire. The fire resistance is the time for which elements performs its functions under specified conditions. These functions may include the ability: not to collapse, to limit the spread of fire, to support other elements. All materials progressively lose their ability to support a load when they are heated. If components of a structure are heated sufficiently, they may collapse. The consequences of such a collapse may vary, depending on how critical the component is in controlling the overall behaviour of the structure. In order to limit the threat that a fire poses to people in a building and to reduce the amount of damage that a fire may inflict, large buildings are divided up into smaller fire compartments using fire resisting walls and floors. Parts of a fire compartment may be divided up by fire resisting construction to protect particular hazard within them.
The performance of fire separating elements may rely heavily on the ability of the structure that supports them to continue to provide that support under fire conditions (Lennon, 1997). The criticality is the degree to which the collapse of an individual structural element affects the performance of the structure as a whole. All main components of a structure are generally expected to exhibit fire resistance proportionate to the nature of the perceived risk. The nature of the risk is usually assessed on the basis of the size and proposed use of the building in which the structural element occurs, which is an important part of a fire safety risk analysis.

The definition of the fire resistance is the ability of construction or its element to satisfy for a stated period of time load bearing capacity, integrity and insulation (same or all of the criteria. As a consequence of European harmonization, fire resistance is increasingly being expressed in terms of $R$, $E$ and $I$. $R$ means the resistance to collapse, i.e. the ability to maintain load-bearing capacity. $E$ is the resistance to fire penetration, i.e. an ability to maintain the integrity of the element against the penetration of flames and hot gases. $I$ is the resistance to the transfer of excessive heat, i.e. the ability to provide insulation from high temperatures.

The term the elements of structure is in fire engineering applied to main structural elements such as structural frames, floors and walls. Compartment walls are treated as elements of structure although they are not necessarily load-bearing. External walls such as curtain walls or other forms of cladding that transmit only self weight and wind loads, and that do not transmit floor loads, are not regarded as load bearing, although such walls may need fire resistance to satisfy other requirements in connection with a need to restrict fire spread between buildings. Load bearing elements may or may not have a fire-separating function. Fire-separating elements may or may not be load bearing.

A fire safety engineering approach takes into account the total fire safety provides a fundamental and economical solution than the prescriptive approaches to fire safety. The modelling of a structure involves three stages. The first stage is to model the fire scenario to determine the heat energy released from the fire and the resulting atmospheric temperatures within the building. The second stage is to model the heat transfer between the atmosphere and the structure. Heat transfer involves three phenomena (conduction, convection and radiation) all contribute to the rise in temperature of the structural materials during the fire event. The third stage is the determination of the response of the structure – basic simple checks, engineering advanced models and sophisticated discrete models based on all data available.

The requirements for the codes contain simple checks, which provide an economic and accessible method in the majority of buildings. For complex problems considerable progress has been made in recent years in understanding how structures behave when heated in fires and in developing mathematical techniques to model this behaviour. It is now possible to predict the behaviour of certain types of structure with a reasonable degree of accuracy. The most common form of analysis is the finite element method (Newman et al. 2000). It may predict thermal and structural performance. In fire, the behaviour of a structure is more complex than at ambient temperatures. Changes in the material properties and thermal movements cause the structural behaviour to become non-linear and inelastic.

In the fire resistance tests the gas temperature is increased to follow a predefined time/temperature curve. This heating regime is very different from that occurring in real fires. The maximum temperature attained in a real fire and the rate at which temperatures increase depend on a number of factors relating to the fuel available, the geometric and thermal properties of the compartment and the availability of openings through which oxygen can be supplied to the fire. Techniques have been developed to mathematically describe a natural fire. The analysis determines the rate at which heat is released from the available fuel, see Schleich et al, (1999). This is a function of the amount of ventilation available and the density and distribution of the fuel itself. Heat loss from the compartment via convection and radiation from the openings, and conduction through the other solid boundaries is calculated before the resulting atmospheric temperatures may be determined. There are the two forms of fire used in standard fire resistance tests. They are timber fires or hydrocarbon fire in petrochemical industry.

The periods of fire resistance specified in regulations attempt to relate the damaging effect of a real fire to an equivalent period of exposure in a standard fire resistance test. Safety factors are introduced to account for building use and height. The modelling of the fire depends upon several factors such as the fire load density, the size and shape of the ventilation openings, and the thermal characteristics of the enclosing compartment. Time equivalence is useful when comparing the performance of an element in a natural fire with the known performance of the same element in a fire resistance test, see Schleich et al, (2001). It is useful for researchers and fire investigators to know that the natural fire was
more masonry is rarely seriously damaged by fire in the sense like other materials can be - it does not buckle like steel, spall like reinforced concrete or burn like timber. On the other hand, the high temperatures can influence the strength of the units their deformations and can destroy the mortar-brick junction and thus can provoke some further reduction in residual compressive strength and sharp drop in flexural and shear strength of the masonry elements exposed to the fire.

2.2 Materials for use in masonry walls
The masonry is strongly composite material consisting of unit, mortar, and concrete infill and reinforcing steel. From that point of view, the fire behaviour of a masonry wall depends on:
- masonry unit material – clay (C), calcium silicate (CS), autoclaved aerated concrete (AAC) or dense/lightweight aggregate concrete (LC);
- type of unit - solid or hollow (type and pattern of holes, percentage of holes), shell and web thickness;
- characteristics of the units – strength, density, porosity;
- type of mortar - general purpose, thin layer or lightweight mortar;
- type of the execution of the pretend (head) and bed joints (fully filled, stripes, unfilled, tongue and groove systems etc.);
- with or without renders (plasters) on one or both sides;
- relationship of the applied load to the resistance of the wall and slenderness of the wall.

2.3 General requirements related to masonry walls
From the point of view of fire protection, a distinction is made between non-load-bearing walls and load-bearing walls and between separating walls (walls along escape ways, walls of stair wells, or compartment walls) and non-separating walls (within single fire compartment).

It is assumed that walls span or extend from one floor to the next floor or to the roof, and that those floors or the roof provide lateral support to the top and bottom of the wall, unless its stability under normal function is achieved by other means, for example buttresses or special ties. Since masonry walls can be either single or multiple leaf elements, a general attention is also focused on the number of leaf, their thickness and if they are load-bearing or not. Also perforated masonry units should not be laid so that the perforations are at right angles to the face of the wall, i.e. the wall should not be penetrated by the perforations.

Special attention has to be paid in designing the joints as well as recesses and chases for fixtures, pipes and cables in the walls.

2.4 Design Practice
Masonry walls themselves do not contribute fuel to the fire, are not subject to flame spread, and do not produce...
smoke or toxic gases in the presence of fire. They do provide solid non-combustible barriers to the spread of fire from the original fire area and can be equally useful in creating safe compartments or escape routes from the building. Therefore, masonry construction is ideally suited to minimizing the potential for ignition of fires and the consequences of fires.

The main fire resistance aspects (Drysdale et al. 1994) of the design of masonry walls relate to the following:

- ability to maintain sufficient load-carrying capacity to support floors and roofs in load-bearing construction
- ability of nonload-bearing firewalls to maintain sufficient strength during and after specified duration of fire to avoid collapse
- thermal characteristics of the wall so as to prevent temperature rise that could possibly cause new ignition
- impact of the failure of other structural elements on the stability of the masonry
- impact of the use of other materials as part of the wall system

2.5 Assessment by testing and tables

While most building codes for other materials relate to both fire prevention and fire protection design procedures, for the masonry this is not a case. Due to the general belief that the masonry is incombustible material little or no investigation were made on development of analytical procedures for assessment of fire resistance of the masonry structures or its components in a real building fire.

All of the current building codes for masonry (EC 6, BS 5628, DIN 4102, UBC, NBC etc.) relate to the fire design of the masonry structures by:

- avoiding premature collapse of the structure (load-bearing function) and
- limiting fire spread (flames, hot gases, excessive heat) and temperature rise beyond designated areas (separation failure).

The requirements for masonry materials for achieving those goals are stipulated by means of numerous tables, which are derived mainly from fire resistance tests of masonry walls with standard temperature-time curve. The tables are drawn up in a standardized form to give consistent information for all masonry material and the standard renderings. Building code requirements for fire resistance typically vary from ½ hours up, depending on type of building, occupancy and criteria of failure (R – load bearing, E – failure due to cracks, holes, etc, and I – insulation failure). Having in mind all variety of materials that can be used in masonry walls it is worth to mention that the values presented in Table 1 reflect only the small portion of those requirements stated in different codes.

Table 1: Ultimate fire Resistance Periods for Load-bearing Clay and Shale Walls in the United States (Drysdale 1994)

<table>
<thead>
<tr>
<th>Ultimate Fire Resistance Period (h)</th>
<th>No-Combustible</th>
<th>Combustible</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

As it concern Eurocodes for masonry structures (ENV1996-1) it is worth to mention that for the moment in the latest version there are no input values in the tables for minimal thickness for fire resistance for different types of masonry (ENV 1996-1, 1995). In the previous version of the EC 6 (ENV 1996-1, 1990) there were some provisional values that were based largely on BS and DIN standards, but with the comment that the method of test according to BS and DIN were not exactly the same. In general the values for given periods of fire resistance are sometimes higher according to the German results while there is some British data that gives a lower thickness that has often been used.

2.6 Assessment by calculation

It has been only recently (Hahn 2001) that first attempts were made to assess the fire resistance of masonry walls by taking into account the relevant failure mode in fire exposure, the temperature dependent material properties and effects of thermal expansions and deformations.

In general the calculation methods may be:

- a simplified calculation model for specific types of walls (analytical solution) and
- global structure analysis for simulating the behaviour of structural masonry elements and the entire structure (FEM solution).

Although there are lack of data for other numerous types of masonry materials, some good results were obtained through simplified calculation procedure both for pillar and wall elements, where in comparison to the experimental results, the results of simplified calculations were a little bit too conservative. Most of the problems that are limiting the usage of both analytical and FEM solution of fire resistance of masonry structures are large variety of materials that have been used in masonry and lack of experimental data for verification of those models. Moreover, it will be quite demanding to prepare a general solution for all types of the masonry. As it can be seen from Figure 1, different types of the units have different behaviour due to thermal actions. Furthermore some types of the units like AAC and CS are even gaining on the strength at some elevated temperatures, while LC and C units are having constant declination of the strength with increasing the temperatures.
3. TIMBER STRUCTURES

3.1 Behaviour of wooden constructions

The protection of buildings against fire is an important topic, which puts constructions made of combustible materials such as wood, and wood based material at a disadvantage (Kollman and Coté 1968). Experiences with devastating fire in the history of our cities led to regulations that forbade the use of wood first as a roofing material and on the ground floor. Afterwards when alternative building materials and design were available wood was more and more replaced especially in urban areas. Today, there is again a tendency to construct multi-storey buildings in wood where the risk of fire plays a key role, see STEP (1995). These buildings require a careful planning with respect to fire protection, which focuses on an organizational, conceptual, constructive and technical approach. Due to the complexity of this topic, the following concentrates on constructive fire protection. Combustibility is an important material property. Fire resistance, however, characterizes a construction element and indicates the period of time it can maintain the load bearing capacity and tightness. Both properties have to be taken into account. Wood and steel are diametrically opposed with respect to these two definitions. Wood burns, however, since charcoal is produced at a constant rate so that the time to failure of wooden construction elements can be easily predicted. Steel is incombustible, however, it has to be protected against heat in order not to loose strength and stiffness. The design of multi-story wood buildings requires good fire resistance, which is in general, achieved by cladding wood structures with incombustible components or, by the use of construction elements especially floors made of solid wood. Floors with a long span have been designed as timber concrete composite, which successfully resisted significantly more than 90 minutes in case of fire.

3.2 Experimental results on floor elements made of solid wood and wood-concrete composite

Combustibility and fire resistance determine the fire behaviour of wood and wooden elements. Instinctively most people are susceptible to attach a high risk to the combustibility of a construction, see Tab. 2. However, if one consider that fire resistance of masonry and wooden building are equivalent, it becomes obvious that the course of a fire depends decisively on the resistance of the wall and floor. Elements from solid wood have a good fire behaviour since there is no heat conduction through hollow spaces. The tightness of the element is crucial and can be achieved by gluing or cladding with other materials. In the past, the fire behaviour of different solid construction elements and whole buildings has been tested. The test results encourage to a further use of wood in the multi-story sector. The following summarizes experimental findings on nailed laminated timber
floors (NLT) that can alternatively be provided with a thick layer of concrete, which guarantees tightness and makes the floor work as a structural composite where the wood is subjected to tension and the concrete to compression. Nailed laminated wood is to be distinguished from glued laminated wood with its tight bond line.

Table 2: Fire resistance of selected solid floor elements and composites

<table>
<thead>
<tr>
<th>No.</th>
<th>Floor and layers</th>
<th>Fire resistance</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NLT (thickness: 110 mm) uncovered</td>
<td>23 min</td>
</tr>
<tr>
<td>2</td>
<td>NLT (thickness: 110 mm) + insulation (mineral: 4 cm) + particle board (16 mm)</td>
<td>35 min</td>
</tr>
<tr>
<td>3</td>
<td>Wood concrete composite NLT (thickness:140 mm) + concrete (thickness: 80 mm)</td>
<td>&gt; 90 min</td>
</tr>
</tbody>
</table>

Figure 4: Wood concrete section on the basis of NLT

4. STEEL STRUCTURES

4.1 Reached Stage

The design methods are used in practice to increase safety of steel structures under fire conditions. New studies have been made in the area of material properties, complex as well simple design models of elements and joints, and fire modelling in last ten years. Very complex and progressive was the research in fire modelling and its application into the design. Among the work done in this field there are two topics design that will be presented here: the lateral torsional buckling of steel I-beams at elevated temperatures and the component model for the behaviour of steel joints in case of fire.

4.2 Lateral-Torsional Buckling of Steel I-Beams

The problem of lateral-torsional buckling of steel I-beams at room temperature is well recognised by practice. The same problem at elevated temperature was studied by Bailey et al., 1996, who uses a three dimensional computer model to investigate the ultimate behaviour of uniformly heated unrestrained beams.

The behaviour of steel I-beams at elevated temperature has been analysed numerically (Vila Real & Franssen, 1999, 2001) leading to a new proposal for the evaluation of its lateral-torsional buckling resistance. This proposal was based on the numerical results from the SAFIR program, a geometrical and materially nonlinear code specially established for the analysis of structures submitted to fire (Franssen, 1995). The capability of this code to model the lateral-torsional buckling of beams has been shown (Vila Real & Franssen, 1999) at room temperature by comparisons against the formulas of ENV 1993-1-1, 1993.

It must be emphasized that the simple model presented here (Vila Real et al. 1999, 2001), was established on the base of numerical simulations using characteristic values for initial out-of-straightness (L/1000) and residual stresses (0.3×235 MPa), which are unlikely to be simultaneously present in a test or in a real fire. In the experimental work we have done, the geometrical imperfections and the residual stresses were measured as well as the nominal yield strength of the material and the Young Modulus. These measured values were used in the numerical calculations.

A set of 120 full-scale tests based on a reaction frame and on a hydraulic system has been carried out for beams of the European series IPE 100 with lengths varying from 0.5 to 6.5 meters. Three tests have been done for each beam length and for each temperature level. The beams were electrically heated by means of ceramic mat elements, heated by a power unit of 70 kVA. To increase the thermal efficiency a ceramic fibre mat has been used around the beam and the heating elements. According to the proposal the design buckling resistance moment of a laterally unrestrained beam, with a class 1 or 2 cross section type, can be calculated by

\[ M_{b,fi, Rd} = \chi_{LT, fi} W_{pl, y, 0, com} f_y \frac{1}{\gamma_{M, fi}} \]  \hspace{1cm} (1)

where \( \chi_{LT, fi} \) is given by

\[ \chi_{LT, fi} = \frac{\phi_{LT, 0, com}}{\phi_{LT, 0, com} + \sqrt{\phi_{LT, 0, com}^2 - (\chi_{LT, 0, com})^2}} \]  \hspace{1cm} (2)

with

\[ \phi_{LT, 0, com} = \frac{1}{2}[1 + \alpha \chi_{LT, 0, com} + (\chi_{LT, 0, com})^2] \]  \hspace{1cm} (3)

and

\[ \alpha = 0.65 \sqrt{235/f_y} \]  \hspace{1cm} (4)

where \( f_y \) is the nominal yield strength of the material, \( W_{pl} \) is the section modulus, \( k_{y, 0, com} \) the reduction factor for yield strength, relative slenderness \( \chi_{LT, 0, com} \), and \( \gamma_{M, fi} \) is the partial safety factor for steel. The lateral-torsional buckling curve now depends on the steel grade as it can be seen in Figure 5.
Both experimental and numerical results have been compared with the simple formulas from ENV 1993-1-2 and the new proposal. The results of these comparisons are shown respectively in Figure 6 and Figure 7. In these figure the regression line is much more close to the ideal continuous line in the case of numerical calculation than for the experimental results.

4.3 A component model for the behaviour of steel joints at elevated temperature

Recent experimental evidence have highlighted the need to evaluate the behaviour of steel joints at elevated temperatures, since they exhibit a distinct change of its moment-rotation response under increasing temperature, that affects the global response of the structure. In terms of cold design, the ‘component method’ constitutes today the widely accepted procedure for the evaluation of the various design values (EN 1993-1-1/A2).

It was purposed an analytical procedure capable of predicting the moment-rotation response under fire conditions (Simões da Silva et al., 2001). This procedure is based in the ‘component method’, that consists of modelling a joint as a extensional springs and rigid links, whereby the springs represent a specific part of a joint make an identified contribution to one or more of its structural properties (component). Each component exhibits a non-linear force deformation response (here taken as a bi-linear approximation), characterised by four properties: elastic stiffness, post-limit stiffness, limit load, yield displacement and limit displacement.

The evaluation of the fire response of steel joints requires the continuous change of mechanical properties of steel (yield stress and Young’s modulus) as temperature increase. In the context of the component method this is implemented at the component level:

\[ F_{iθ} = k_{yθ} F_{i;20°C} \]
where \( F \) is force in component, \( K \) is stiffness of the component, \( k_{E,\theta} \) is reduction factor relative to \( E \), \( k_{\gamma,\theta} \) is reduction factor relative to \( f_{\gamma} \), and \( \delta \) is deformation of component. Introducing these equations in any evaluation of moment-rotation response of steel joints at room temperature yields the required fire response:

\[
M_{i,\theta} = k_{\gamma,\theta} M_{i,20^\circ C};
\]

\[
\frac{\theta_{i,\theta}}{S_{i,\theta}} = \frac{F_{i,\theta}}{K_{i,\theta}} = \frac{k_{\gamma,\theta}}{k_{E,\theta}} \delta_{i,20^\circ C},
\]

\[
S_{i,\theta} = E_{i,\theta} \frac{z^2}{\sum_i k_{i,\theta}} = k_{E,\theta} S_{i,20^\circ C};
\]

where \( \theta \) is rotation of joint, \( M \) is bending moment moment, and \( S \) is the joint stiffness. This procedure may be applied to a fire event, characterised by any fire-load curve: anisothermal, isothermal or transient situation. An application to a cruciform flush end-plate beam-to-column steel joint is presented and compared to the experimental results obtained under various loading conditions (Al-Jabri et al. 1997). Comparison with experimental results has shown good agreement with the proposed methodology, as represented in Figure 8. The fire response of steel joints involves additional complexity to the corresponding cold analysis. This analytical methodology greatly simplifies this problem, allowing a direct solution from the knowledge of the response at ambient temperature.

5.2 Thermal and mechanical properties of aluminium

Features of aluminium alloys are changing under exposure at high temperatures. Relevant values of both physical and mechanical properties are given in the European Prestandard ENV 1999-1-2 (1998). The first ones allow the temperature distribution in the cross-section to be determined by thermal analysis, while the second ones allow the bearing capacity of the structure at elevated temperature to be evaluated by means of structural analysis. In general, it has to be emphasised that almost all the material properties are dependent on the type of alloy, but some common features may be identified, allowing some general trends to be recognised.

In particular, it has to remarked that: (1) thermal elongation of aluminium is about 2,5\( \cdot \)10\(^{-5}\) °C\(^{-1} \), therefore,
more than twice that of steel; (2) specific heat of aluminium alloys ranges from 0.9 kJ/kg °C (at room temperature) to 1.1 kJ/kg °C (at 500 °C), therefore about two times the one of steel (nevertheless due to the ratio of specific weight between steel and aluminium, for a given cross section and fire exposure, the former has an higher volumetric heat capacity than the latter, resulting in more rapidly heat up of aluminium structures than steel structures); (3) unlike steel, thermal conductivity of aluminium is increasing with increasing temperatures and ranges from 140-190 W/m°C at room temperature to 180-220 W/m°C at 400 °C, it being therefore more than three times higher than for steel; (4) the emissivity of aluminium surface is dependent on the surface finish, making it difficult to be determined accurately (suggested values are 0.3 and 0.7 for clean uncovered surfaces and for painted and covered surfaces, respectively, while for steel the suggested value is 0.625).

From the structural design point of view, the most important difference between aluminium and carbon steel is the shape of the stress-strain curve. In fact, in case of aluminium, the stress-strain curve is non-linear and there is no well-defined yield stress due to continuous hardening. Besides, it has to be considered that as a function of both the chemical composition of the alloy and of the possible type of applied heat-treatment, there is a great variety of aluminium alloys having mechanical features substantially different from each other in terms of limit strength, ductility and shape of the stress-strain curve. On the other hand, the mechanical properties of aluminium at transient high temperatures are complex and cannot be easily generalised. A wide preventive determination of experimental data and accurate material models is therefore necessary for an accurate and reliable determination of load-bearing capacity of aluminium structures under fire conditions.

The main parameters for characterising the mechanical behaviour of aluminium alloys in the elastic range are Young's modulus (E) and 0.2% proof strength (f_0.2), the latter being used as conventional elastic limit of the material. The former is practically independent from the adopted treatment, while the latter is related to the type of alloy under consideration. In Figure 1, values suggested by Eurocode 9 for typical aluminium alloys and tempers at elevated temperatures are depicted. It can be observed the remarkable variation of the conventional elastic limit with temperature (f_0.2,T) due to type of alloy and temper. For instance, relative strength values at 200 °C are 90 and 40 percent for 5052-O and 7075-T6 alloy, respectively. Even though there is any possibility to gather typical common trend among different groups of alloy, it can be noted that the beneficial effects of heat treatment and work hardening, used to improve the mechanical properties at normal temperatures, diminishes at high temperatures.

On the other hand Eurocode 9 (ENV 1999-1-1) does not supply any further information regarding the material behaviour in the plastic range, not allowing for taking into account at elevated temperatures the rather important contribution provided by the hardening features of aluminium alloys.

Several additional data gathered by test results, including ultimate strength (f_u) and ductility (e_u) of typical aluminium alloys are provided in ASM (1994). From the analysis of relevant results, the following remarks may be drawn: (1) for heat treated alloys (T) and work hardened alloys (H) the variation of hardening ratio (f_u/f_0.2) with temperature is slight, while it is considerable for alloys in the annealed stage (O); (2) the inelastic behaviour of the alloy is independent from the adopted treatment for temperatures higher than 200 °C; (3) there is a remarkable increase of uniform strain with increasing temperature for all types of alloys; (4) Inelastic features of aluminium alloys are strongly affected by the time of exposure at elevated temperature, showing higher ductility and lower ultimate strength as far as the exposure time increases, see Conserva et al. (1992). Characteristic temperature dependence curves of the above parameters for some typical alloys are shown in Figure 10, permitting some of the above conclusions to be stressed.
5.3 Ongoing research projects in Europe

Many issues related to the structural behaviour of aluminium exposed to fire have to be still solved in order to allow the designer to apply accurate and rational procedure for the safety assessment of members and structures. Such aspects are concerned with both more realistic material modelling to be used for global structural analysis and the actual response of aluminium members at elevated temperatures.

The behaviour of aluminium in fire is presently under investigation at University of Naples Federico II. The general scope of the research activity is the definition of reliable rational methods for structural fire design. Therefore, mechanical models able to account for the material behaviour at elevated temperature in both elastic and inelastic ranges have been proposed. Such models are based on the well known Ramberg-Osgood law (Mazzolani 1995), whose parameters are given in a closed-form as a function of the corresponding values at room temperatures and specific temperature-dependent relationships, which have been calibrated on the basis of existing test results (Ponticelli, 1999). In cooperation with University of Liege, the above material model has been introduced into the well known computer program SAFIR (Franssen et al. 1995), allowing the behaviour of complex aluminium structures exposed to standard fires to be predicted up to plastic collapse by means of an accurate but direct and quite simple approach.

Wide research activities are also carried out at the NTNU of Thondheim. In particular the behaviour of rectangular hollow cross-section columns made of 6082 aluminium alloy (both temper T4 and T6) subjected to fire has been recently investigated by Langelle and Amdahl (1998) by means of both experimental tests and complex numerical analyses. Several interesting conclusions are drawn in the above study, emphasizing the necessity to undertake further research activities to obtain more comprehensive database and a better understanding of some fundamental phenomenological aspects which have been evidenced by tested aluminium alloys contrarily to the simplified schematisation suggested by the present design code. In particular, some of the important issues arisen are: (1) creep effects are irrelevant when constant heating rate is applied, while they are important in case of tests carried out under constant temperature; (2) EC9 yields reasonable capacities for tested columns, except for those tested with constant heating rates, due to the fact that creep effects are not correctly taken into account in such a condition; the difference between experimental and design capacities depends on both the type of temper and the fabrication process, it being larger in case of welded columns and temper T6; (4) reduction of yield strength of AA 6082 at elevated temperatures is different for temper T4 and T6, while design codes give similar reduction values for both tempers (in particular T4 exhibited a less pronounced loss of capacity due to the artificial ageing caused by the applied heating); (5) The strength of the alloy is invariant with respect to temper at 285 °C, and the detrimental effect of welding completely vanishes at this temperature.

6 CONCLUSION

The European knowledge of fire design brings good engineering tools for modelling of the structural behaviour under fire conditions. Three steps of procedure may be separated: the model of fire scenario in the compartment, the heat transfer to the structure, and the response of the structure. For all steps are ready for use the design models. The simplest design models are supported by design charts and design rules. The engineering models are based on natural fires scenario, refined transfer of the heat between the atmosphere and the structure and non-linear global analyses. The complex models, based on the FE modelling of fire scenario and 3D non-linear behaviour of structure, are ready to be applied for prediction of the structural behaviour under exceptional fire loading. All design and knowledge steps are under development by the experimental, numerical and analytical modelling to address and to reach the tomorrow asked level of safety.

7 REFERENCES


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