FIRE ENGINEERING OF STEEL STRUCTURES

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Abstract
The paper summarizes the reached knowledge in the field of design of steel and composite structures under fire action. There are a number of approaches to ensure the safe design of structures under fire conditions. These range from a simple elemental prescriptive approach to a more advanced structural fire engineering approach. In the simple approach, realistic structural and fire behaviour are ignored and optimum design solutions in terms of safety and economy may not be reached. By considering the actual fire and structural behaviour, through more advanced methods, any weak-links within the design can be identified, and rectified, allowing safer, more robust, and possibly more economical buildings to be constructed. This paper presents the state of the art of fire safety for steel and composite steel and concrete buildings, introduces the benefits of using fire engineering (performance – based) approaches, shows the progress in design of connections exposed to fire, and summarises the latest European materials which supports the design, and which is freely available on internet.

Key Words
Structural steel, fire design, connections, floor plates, fire protection, Eurocodes, internet.

1 INTRODUCTION

Existing recommendations, models and regulations about structural safety are mostly aimed at ensuring an adequate level of safety for constructions under normal loading conditions. The overall structural safety is dealt by assuming an increase in the multipliers of service loads up to reach the collapse values. This approach has led to a satisfying degree of accuracy in the prediction of 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 it is possible to achieve a good response of structures subjected to random actions. These approaches form the basis of most recent developments in the field of regulations and are part of almost all relevant structural codes, including the Eurocodes of course, where specific allowance for accidental loading conditions is made.

For member states of the European Union, safety requirements in case of fire are based on the Construction Products Directive, Council Directive 89/106/EEC, from 21.12. 1988, see [1]. The Directive shall be applied to construction products as the essential requirement in respect of construction works. In Annex I of the Directive, the essential requirements are summarised for mechanical resistance and stability in the first paragraph and for fire safety in the second one. The construction works must be designed and built in such a way that in the event of an outbreak of fire: the load-bearing capacity of the construction can be assumed for a specific period of time; the generation and spread of fire and smoke within the works are limited; the spread of the fire to neighbouring construction works is limited; occupants can leave the works or be rescued by other means; the safety of rescue teams is taken into consideration. The load-bearing capacity of the construction may be modelled on the principles summarised in the various fire parts of structural Eurocodes.

2 FIRE DESIGN

2.1 Fire resistance

Fire resistance is commonly used to characterize the performance of elements of structure in fire. In this regard, fire resistance may be defined as the time for which elements of a structure satisfactorily perform their required functions under specified fire 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, see [2]. 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 fire resistance is the ability of construction or its element to satisfy for a stated period of time the load bearing capacity, integrity and insulation, separately or combined. As a consequence of European harmonization, fire resistance is increasingly being expressed in terms of *R*, *E* and *I*, where *R* means the resistance to collapse, i.e. the ability to maintain load-bearing capacity; *E* is the resistance to fire penetration, i.e. the ability to maintain the fire integrity of the element against the penetration of flames and hot gases; and *I* is the resistance to the transfer of excessive heat, i.e. the ability to provide insulation to limit excessive temperature rises. The term elements of structure is used in fire engineering to mean 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.

2.2 Fire design

The design for fire safety have traditionally followed prescriptive rules and may now apply fire engineering (or performance based) approaches, examples of which are given in the various structural fire standards Eurocodes in documents EN 1990: 2002 and 1991-1-x: 2005, see [3] - [6]. A fire engineering approach takes into account fire safety in its entirety and provides a more fundamental and economical solution than the prescriptive approaches. Within the framework of fire engineering approach, designing a structure involves four stages. The first stage is to model the fire scenario to determine the heat 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 conduction, convection and radiation which all contribute to the rise in temperature of the structural materials during the fire event. The third stage evaluates the mechanical loading under fire conditions, which differs from the maximum mechanical loading for ambient temperature design, due to reduced partial safety factors for mechanical loading under fire. The fourth stage is the determination of the response of the structure at elevated temperature.

The design recommendations in codes contain simple checks, which provide an economic and accessible method for 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 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. 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.

2.3 Fire modelling

In the standard fire resistance tests the gas temperature is increased to follow a predefined time/temperature curve, called according to Eurocodes the standard nominal fire curve or earlier ISO 834 fire curve. This heating regime is 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 related 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 [7]. This is a function of the amount of ventilation available and the density and distribution of the fuel itself. Heal 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.

Fire models of various degrees of sophistication may be used to obtain a design fire scenario. At the simplistic level, periods of standard fire resistance are specified in regulations. The next level up is to attempt to relate the damaging effect of a real fire to the standard fire by using the time-equivalent approach. Ideally, the equivalent time should be based on comparing the performance of an element in a natural fire with the known performance of the same element in a fire resistance test. The time equivalency approach is attractive to fire investigators and fire engineers because this allows them to relate the complex behaviour of a real fire to the standard fire resistance, which is a well understood concept. An equivalent time equation is given in Eurocode 1 Part 1.1 which expresses the equivalent time as a function of the fire load, ventilation and thermal characteristics of the enclosure, see [4].

A more rational, and still relatively simple, approach for a post-flashover real fire is to assume uniform temperature within the fire enclosure and to specify the uniform fire temperature – time relationship. Eurocode 1 Part 1.1 refer to them as parametric fire curves and provides equations to calculate these curves using the three aforementioned parameters, which is based on the pioneering research work of Pettersson, see [8]. At the other end of complexity of fire modelling, computational fluid dynamics modelling FCD may be used, see [9].

2.4 Structural response

Structural response and its modelling under fire condition depend on the applied structural materials as well as the extent of the modelled structure which may be the whole structure or its parts or individual elements. Standard fire resistance tests can only provide limited guidance. As far as different materials are concerned, aluminium and steel transfer heat rapidly. Timber, masonry, concrete and lightweight concrete have better insulation properties, see [10] and [11]. The added insulation may be economical for aluminium, steel and timber structures. The simplified design models in codes such as Eurocodes are mostly based on design check equations for ambient temperature design, see [12]. On the other hand, more advanced models of global analysis using finite element method may be used to deal with structural interactions between different structural members and connections as well as structural behaviour at large deformations.

3 SOLUTIONS

3.1 Application of fire protection

The traditional and still the most popular approach to achieve the required levels of safety is to apply fire protection to all exposed areas of steel. The use of fire protection can be in the form of proprietary materials comprising sprays, boards or intumescent coatings, or generic materials comprising concrete block, gypsum plaster and certain types of plasterboard. The use of intumescent coatings has increased recently, especially when applied off-site which reduces construction time and arguably increases quality. Typically the specification of fire protection thicknesses to steel elements has been based on ensuring that the steel does not exceed a maximum temperature of 550 °C for columns, and 620 °C for beams supporting concrete floors, for a given fire resistance period tested in a standard furnace. These temperatures are based on the assumption that a fully-stressed member at ambient conditions will lose its design safety margin when it reaches 550 °C. The maximum temperature for beams supporting concrete floors is increased to 620 °C, since the top flange is at a lower temperature compared to the web and bottom flange, due to the concrete floor acting as a heat sink. Generally the 550/620 °C maximum temperatures are considered conservative since the members are not fully stressed at ambient temperature, the stress-strain-temperature relationship of steel at elevated temperatures, used to derive the 550/620 °C values, is too simplistic, and in practice the structural elements do not behave in isolation. It is possible to reduce the protection thickness based on the design of steel members during the fire limit state using current codes of practice. However, this has been found to be very difficult in practice due to the reluctance of protection manufacturers to release the required thermal properties of their materials. It is possible to work with individual manufacturers to on thicknesses based on the fire design specify protection of the steel members on a project-by-project basis, but this increases the design time and is rarely a method adopted by designers, see [13].

3.2 Partial fire protection

It is possible to adopt forms of construction, see [14], which eliminate the need for additional passive fire protection in multi story as well as single story buildings. The common forms of beams and columns that utilise partial protection are described below. The construction systems have generally been developed based on standard fire resistance tests, EN 1363-1:1999, see [15], and the basic principles given in fire design codes EN 1993-1-2: 2005 and EN 1994-1-2: 2005. By placing a significant portion of the beam within the depth of the supported concrete slab it is possible to specify steel beams without the need to apply additional fire protection. The slim-floor beams systems, are constructed such that the beam is encased in the supporting concrete slab with only the bottom flange or plate exposed to any fire. SCI and Corus have promoted systems known as Slimflor and Slimek where beams can readily achieve 60 min fire resistance without the need to protect the exposed bottom flange or plate. The Slimdek system, incorporating an asymmetrical beam, has been tested at full-scale and shown to perform extremely well when subjected to a severe fire.

Another form of construction, where the beam is partially protected by concrete, consists of filling the area between the flanges and web with reinforced concrete. This type of construction is popular in continental Europe where the cost of proprietary fire protection materials is high. The system can readily achieve R120 fire resistance and has the advantage of being resistant to impact damage, although the increase in self-weight of the structure and buildability issues can be seen as a disadvantage.

3.3 Floor plates

The Building Research Establishment BRE has developed a simple structural model, see [16], that combines the residual strength of the steel composite beams with the slab strength calculated using a combined yield line and membrane action model for multi-storey composite buildings. The slab model is based on catenary action of the slab and the resistance of the unprotected composite beam, see [17], The model was applied to the concrete slabs as well. The critical parts of the model are the ductility of the mesh and the assumption of the supported boundaries of the slab. The model has been calibrated against the Cardington fire tests, see Fig. 1, and other test results on slabs. Fire tests undertaken in recent years on the eight-storey steel framed building at BRE's Cardington Laboratory have demonstrated the inherent fire resistance of composite flooring systems. This improved performance is the result of membrane action developing in the lightly reinforced concrete slab enabling it to bridge over its fire damaged supporting steel beams and safely carry the applied load to the columns.

However, none of the floors tested at Cardington failed and without data on the mode of failure neither the capacity of the composite floor slabs nor the margins of safety associated with the design methods can be established, see [18]. E.g. during the sevenths Cardington test the applied load was 6,1 kNm^{-2} and the predicted resistance by catenary action 4,1 kNm^{-2}, but collapse was not reached, see [19]. Therefore a compartment fire test on the steel framed building with sufficient applied load allows the actual mode of failure to be determined so as to evaluate the accuracy of existing design guidance.

Fig. 1 Compartment after the seventh large Cardington test, see (Wald et al, 2006), the slab residual deformation 915 mm

4 CONNECTIONS AT ELEVATED TEMPERATURE

Failure of the WTC on 11th September 2001 alerted the engineering profession to the possibility of connection failure under fire conditions. The failure of the connections is thought, by some, to have initiated the progressive collapse of both towers.

When subject to fire, steel loses both its strength and stiffness. Steel structures also expand when heated and contract on cooling. Furthermore the effect of restraint to thermal movement can introduce high strains in both the steel member and the associated connections, see [20]. EN 1993-1-2 gives two approaches for the design of steel connections. In the first approach fire protection is applied to the member and its connections. The level of protection is based on that applied to the connected members taking into account the different level of utilisation that may exist in the connection compared to the connected members. Fire tests on steel structures have shown that the temperature within the connections is lower compare to connecting steel members. This is due to the additional material around a connection, column, end-plate, concrete slab etc., which significantly reduces the temperatures within the connections compared to those at the centre of supported beam.

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. Traditionally steel beams have been designed as simply supported. However it has been shown in recent large scale fire tests on the steel building at Cardington, see [21], see Figs 3 and 4, in real fires and in experimental results on isolated connections, that joints that were assumed to be pinned at ambient temperature can provide considerable levels of both strength and stiffness at elevated temperature. This can have a beneficial effect on the survival time of the structure.

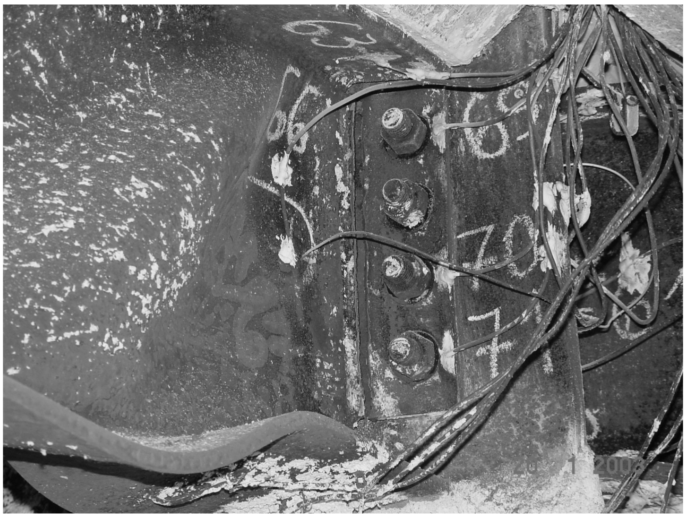

Fig. 3 Rupture of the end plate beam to column connection without lost of the bearing resistance during the seventh large Cardington fire test, see [20]

A more detailed approach uses an application of the component approach together with a method for calculation the behaviour of welds and bolts at elevated temperature. By using this approach the connection moment, shear and axial capacity can be evaluated at elevated temperature, see [22] and [23]. In terms of cold design, the component method constitutes today the widely accepted procedure for the evaluation of the various design values. It has now been validated as an analytical procedure that is capable of predicting the moment-rotation response under fire conditions. This procedure consists of modelling a joint as extensional springs and rigid links, whereby the springs represent a specific part of a joint making an identified contribution to one or more of its structural properties, component. Each component exhibits a non-linear force deformation response, characterised by three main properties: elastic stiffness, design resistance and deformation capacity. At elevated temperature the influence of the normal forces needs to be taken into account.

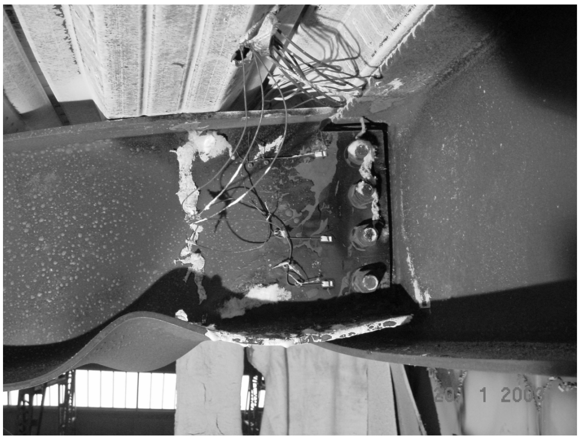

Fig.4 Deformation of the fin plate beam to column connection during the seventh large Cardington fire test, see [20]

European standard for fire design EN 1993-1-2:2005 [5] gives two approaches for the design of steel connections. In the first approach fire protection is applied to the member and its connections. The level of protection is based on that applied to the connected members taking into account the different level of utilisation that may exist in the connection compare to the connected members. A more detailed approach is used in the second method which uses an application of the component approach in EN 1993-1-8:2005 [5] together with a method for calculation the behaviour of welds and bolts at elevated temperature. By using this approach the connection moment, shear and axial capacity can be evaluated at elevated temperature. Traditionally steel beams have been designed as simply supported. However it has been shown in recent large scale fire tests on the steel building at Cardington, in real fires, and in experimental results on isolated connections, that joints that were assumed to be pinned at ambient temperature can provide considerable levels of both strength and stiffness at elevated temperature. This can have a beneficial effect on the survival time of the structure.

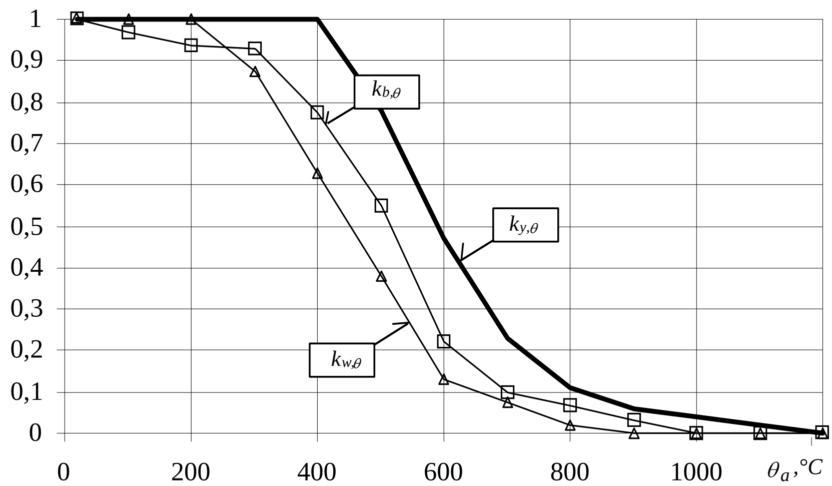

Fig. 5 Reduction factor $k_{\mathrm{b},\theta}$ for bolt resistance, $k_{\mathrm{w},\theta}$ for weld resistance and $k_{\mathrm{y},\theta}$ for yield strength, EN 1993-1-2:2005 [5]

Experimental studies have shown that the strength and stiffness of a bolt reduces with increasing temperature. In particular they show a marked loss of strength between 300 ºC and 700 ºC. The results of this work have been included in EN 1993-1-2 [5], where $k_{b,\theta}$ is used to describe the strength reduction with elevated temperature. $k_{b,\theta}$ is given in Fig. 5. The shear resistance of bolts in fire may be evaluated using the following expressions

$$F_{v,t,Rd} = F_{v,Rd}\ k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}} \tag{1}$$

where γ_M is the partial safety factor for the resistance and $\gamma_{M,fi}$ is the partial safety factor for fire. The bearing resistance of bolts in fire may be predicted using

$$F_{b,t,Rd} = F_{b,Rd}\ k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}} \tag{2}$$

and the tension resistance of a single bolt in fire is given by

$$F_{ten,t,Rd} = F_{t,Rd}\ k_{b,\theta} \frac{\gamma_m}{\gamma_{m,fi}} \tag{3}$$

The question consists of two parts: calculating the temperature distribution in the joints and calculating the weld resistance at high temperature. The design strength of a full penetration butt weld, for temperatures up to 700ºC, should be taken as equal to the strength of the weaker part of the joint using the appropriate reduction factors for structural steel. For temperatures higher than 700ºC the reduction factors given in EN 1993-1-2:2005 for fillet welds can be applied to butt welds. Design strength per unit length of a fillet weld in a fire may be calculated as

$$F_{w,t,Rd} = F_{w,Rd}\ k_{w,\theta} \frac{\gamma_m}{\gamma_{m,fi}} \tag{4}$$

The thermal conductivity of steel is high. Nevertheless, because of the concentration of material within the joint area, a differential temperature distribution should be considered within the joint. Various temperature distributions have been proposed or used in experimental tests by several authors. According to EN 1993-1-2:2005 [2], the temperature of a joint may be assessed using the local massivity value *A/V* of the joint components. As a simplification, a uniform distributed temperature may be assumed within the joint; this temperature may be calculated using the maximum value of the ratios *A/V* of the adjacent steel members. For beam-to-column and beam-to-beam joints, where the beams are supporting any type of concrete floor, the temperature may be obtained from the temperature of the bottom flange at mid span.

Applying the expressions referred to in EN 1993-1-2:2005 [5], see Fig. 6, the temperature of the joint components may be determined as follows:

The depth of the beam is less than 400 mm

$$\theta_h = 0{,}88\,\theta_0\left[1 - 0{,}3\left(a/h\right)\right] \tag{5}$$

where θ_0 is the temperature of the lower beam flange at mid span. The depth of the beam is greater than 400 mm

$$\theta_h = 0{,}88\,\theta_0 \quad \text{a is less than } h/2 \tag{6}$$

$$\theta_h = 0{,}88\,\theta_0\left[1 + 0{,}2\left(1 - 2a/h\right)\right] \quad \text{a is greater than } h/2 \tag{7}$$

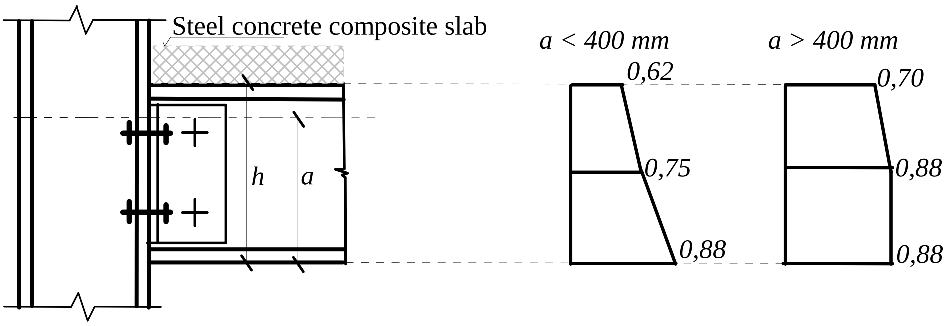

Fig. 6 Thermal gradient within the depth of connection

The influence of the prediction of the temperature to the accuracy of the modelling is relatively high as is shown on the comparison of calculated temperatures of the end plate connection measured during the Ostrava fire test, see Fig. 7 [26], which was performed at 2006 on the building before demolition. The sensitivity of the prediction may be expressed by the reduction of the resistance of bolts, see Fig. 5. The bolt resistance decrease in 45 min of fire to 19 % in case of prediction by lower flange temperature and to 6 % by prediction from section factor of connected beam, but the reduction to 48 % only was evaluated based on the measured temperature, see Fig. 8.

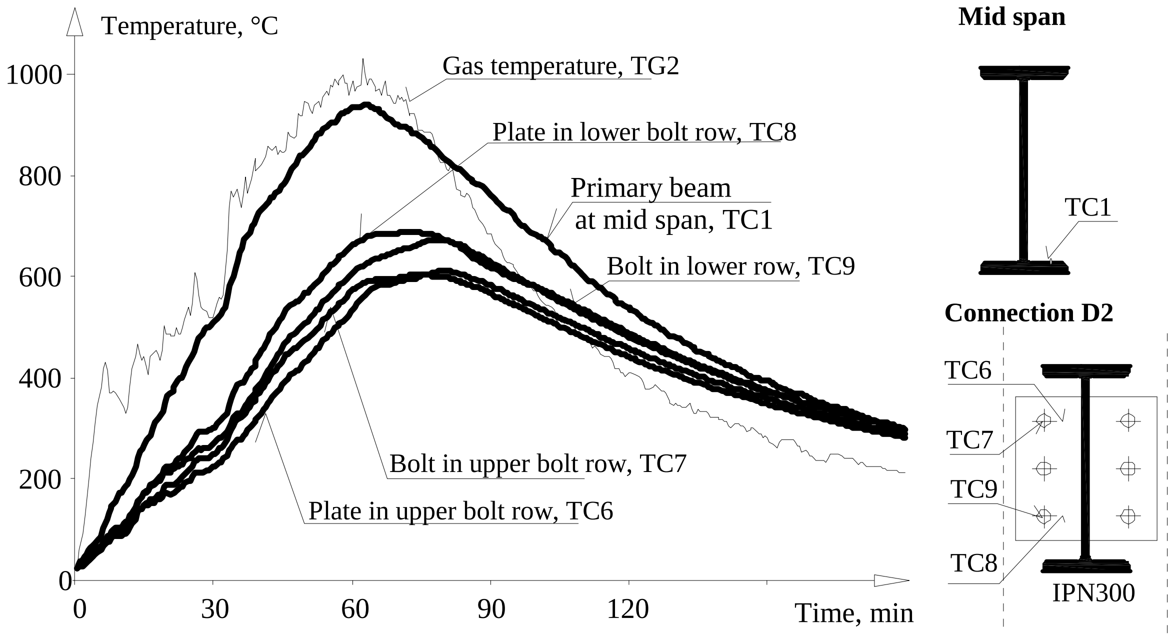

Fig. 7 Measured temperatures over the height of beam-to-column header plate connection during the Ostrava fire test [26]

The component method, see [24] and EN 1993-1-8: 2005 [25], that consists of the assembly of extensional springs and rigid links, may be adapted and applied to the evaluation of the behaviour of steel joints under elevated temperatures. Depending on the objective of the analysis, a simple evaluation of resistance or initial stiffness may be pursued or, alternatively, a full non-linear analysis of the joint may be performed, taking into account the non-linear load deformation characteristics of all the joint components, thus being able to affect the moment-rotation response, see Fig. 9.

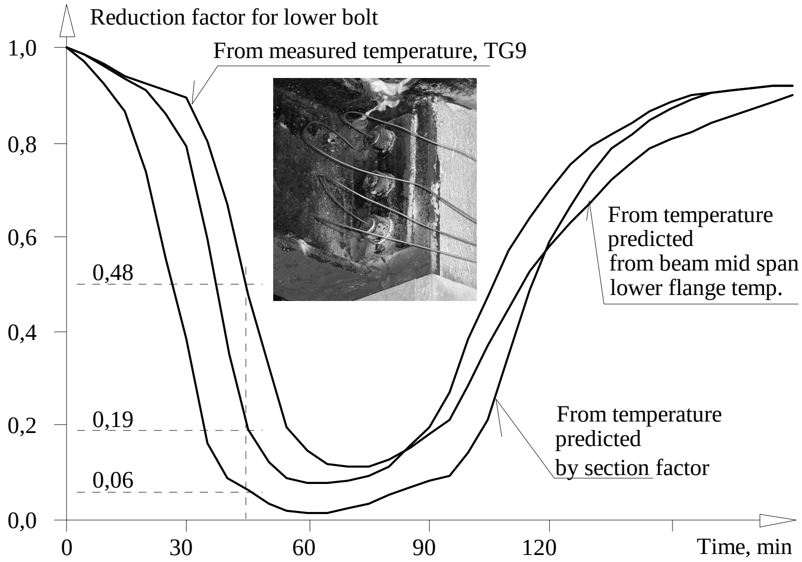

Fig. 8 Reduction of the resistance of bolts in the lower row of the connection A2 according to the different models compared to the reduction obtained from the measured temperature

To evaluate the non-linear response of steel joints in fire, knowledge of the mechanical properties of steel with increasing temperature is required. In the context of the component method, this is implemented at the component level. The elastic stiffness, K_e, is directly proportional to the Young's modulus of steel and the resistance of each component depends on the yield stress of steel. Eq. (8) to (10) illustrate the change in component force-deformation response with increasing temperature for a given temperature variation θ of component i.

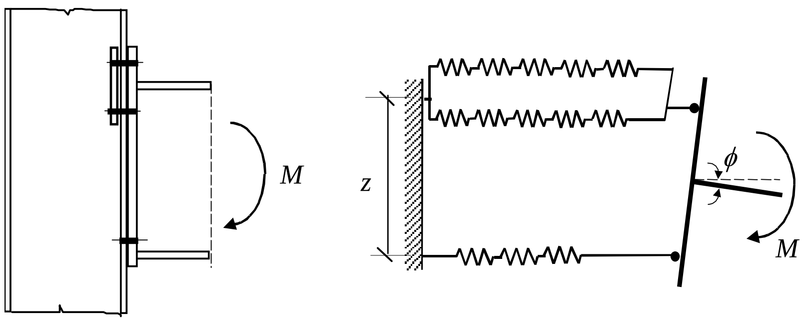

Fig. 8 Component method applied to a typical beam-to-column joint,
a) joint, b) component model

$$F_{i,y,\theta} = k_{y,\theta}\ F_{i,y,20°C} \tag{8}$$

$$K_{i,e,\theta} = k_{E,\theta}\ K_{i,e,20°C} \tag{9}$$

$$K_{i,pl,\theta} = k_{E,\theta}\ K_{i,pl,20°C} \tag{10}$$

Introducing Eqs (8) to (10) for the corresponding values of K_e, K_{pl} and F_y in any evaluation of moment-rotation response of steel joints at room temperature yields the required fire response. Implementation of this procedure allows the fire resistance to be established in any of two domains: the resistance, by finding the reduced resistance at design temperature, and the temperature, by observing the critical temperature for loading to be compared with design temperature.

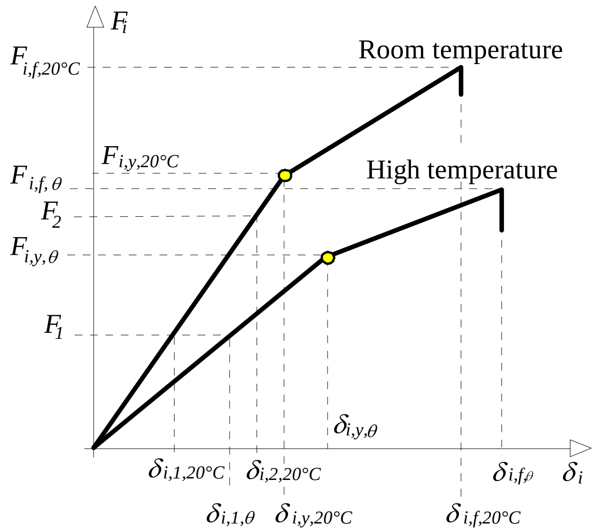

Fig. 9 Isothermal force-deformation response of component

With reference to Fig. 9, for a given level of applied force F, the component deformation $\delta_{i,\theta}$ is given

$$\text{for } F' < F_{i,y,\theta} \text{ as } \delta_{i,\theta}(F_1) = \delta_{i,1,\theta} = \frac{F_1}{K_{i,e,\theta}} = \frac{F_1}{k_{E;\theta}\, K_{i,e,20°C}} = \frac{1}{k_{E,\theta}}\, \delta_{i,20°C}(F_1) \tag{11}$$

$$\text{for } F = F_{i,y,\theta} \text{ as } \delta_{i,y,\theta} = \frac{F_{i,y,\theta}}{K_{i,e,\theta}} = \frac{k_{y,\theta}}{k_{E,\theta}}\, \delta_{i,y,20°C} \tag{12}$$

$$\text{for } F_2 \geq F_{i,y\theta} \text{ as } \delta_{i,\theta}(F_2) = \delta_{i,2,\theta} = \delta_{i,y,\theta} + \frac{1}{k_{E,\theta}}\, \frac{\delta_{i,f,20°C} - \delta_{i,y,20°C}}{F_{i;f,20°C} - F_{i,y,20°C}} \left(F_2 - F_{i,y,\theta}\right) \tag{13}$$

From equilibrium considerations, the bending moment for a given level of joint deformation is given by

$$M_\theta = \underset{(r=1,2)}{F_{r,\theta}}\; z = k_{y,\theta}\; M_{20°C} \tag{14}$$

Similar expressions can be derived for stiffness and rotation of the joint, and the initial stiffness of a joint loaded in bending, at temperature θ is given by

$$S_{1,\theta} = \frac{E_\theta\, z^2}{\sum_i \frac{1}{k_{i,\theta}}} = k_{E,\theta} \times S_{1,20°C} \tag{15}$$

The rotation at yield of the component i follows from

$$\phi_{i,y,\theta} = \frac{M_{i,y,\theta}}{S_{i,y,\theta}} = \frac{k_{y,\theta}\, M_{i,y,20°C}}{k_{E,\theta}\, S_{i,2,20°C}} = \frac{k_{y;\theta}}{k_{E;\theta}}\, \phi_{i,y,20°C} \tag{16}$$

Eqs (11) to (16) give the generic moment-rotation curve at a constant temperature θ where the yielding sequence of the various components is identified. For a joint under uniform temperature distribution, the critical temperature is defined as the maximum temperature of the joint corresponding to failure of the joint,

$$M_{j,Sd} = M_{j,max,\theta} \tag{17}$$

According to EN 1993-1-2:2005 [5] the evaluation of the critical temperature requires the calculation of the degree of utilization of the joint at time $t = 0$, μ_0, defined as the relation between the design effect of the actions for the fire design situation and the design resistance of the steel member, for the fire design situation, at time *t*. For the present case of steel joints, the degree of utilization is explicitly given by:

$$\mu_0 = \frac{M_{j,Sd}}{M_{j,max,20°C}} \tag{18}$$

Using Equation (17) allows the direct calculation of the critical temperature of the joint from Eq. (9.28) in the standard EN 1993-1-2: 2005 [5],

$$\theta_{cr} = 39{,}19 \ln\left[\frac{1}{0{,}967\mu_0^{3{,}833}} - 1\right] + 482 \tag{19}$$

5 DESIGN SUPPORTS ON INTERNET

On internet is in the field of fire engineering available the educational material of a RFCS dissemination project DIFISEK (DIssemination of structural FIre Safety Engineering Knowledge), see Fig. 10. The PowerPoint presentations, see Fig. 11, as well as lecture notes, see Fig. 12, explain the fire engineering approach, comprising the whole spectrum of fire engineering from the calculation of gas temperatures to the design of structural elements in order to resist fire. The prescriptive rules with standard fire curve as well as the real behaviour of a fire are covered. All lecture and presentation materials developed under DIFISEK are in accordance with the Eurocodes. The documents were translated into German, French, Spanish, Dutch and Finnish languages and till the end of 2008 will be available also in Czech, Estonian, Greek, Hungarian, Italian, Lithuanian, Polish, Portuguese, Romanian, Slovenian, and Swedish. In addition, a database dealing with fire design software is provided.

All language versions as well as the database and the freely available software are included on a CD-ROM and are also available for free download at URL www.difisek.eu, see [27]. Both are organised through a user-friendly menu tool in HTML. Information has been grouped into the parts: Thermal and mechanical actions, Thermal response, Mechanical response, Software for fire design, Worked examples, and Completed projects. The actions from the occurrence of fire until the eventual collapse of the structure are represented and subdivided into the Parts 1 to 3. In Part 4 existing fire design software is analysed, validated and explained and in Worked examples according to Eurocodes.

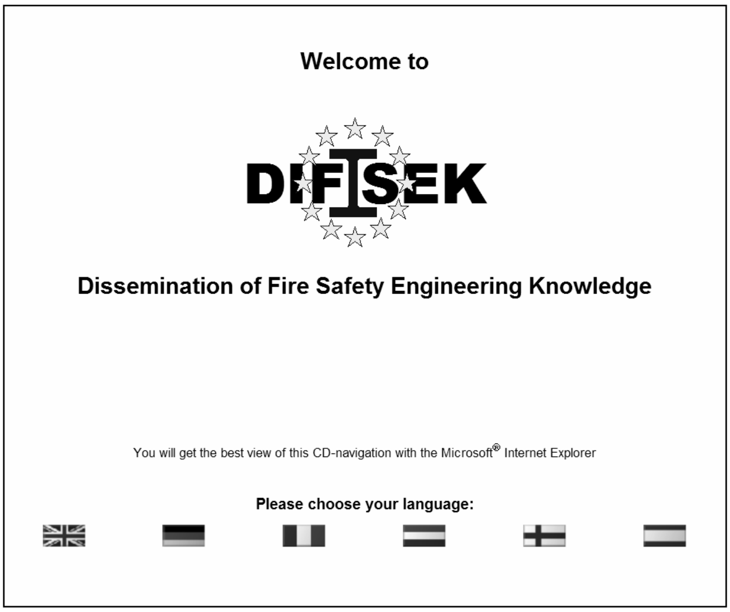

Fig. 10 Home page of DIFISEK, see [27]

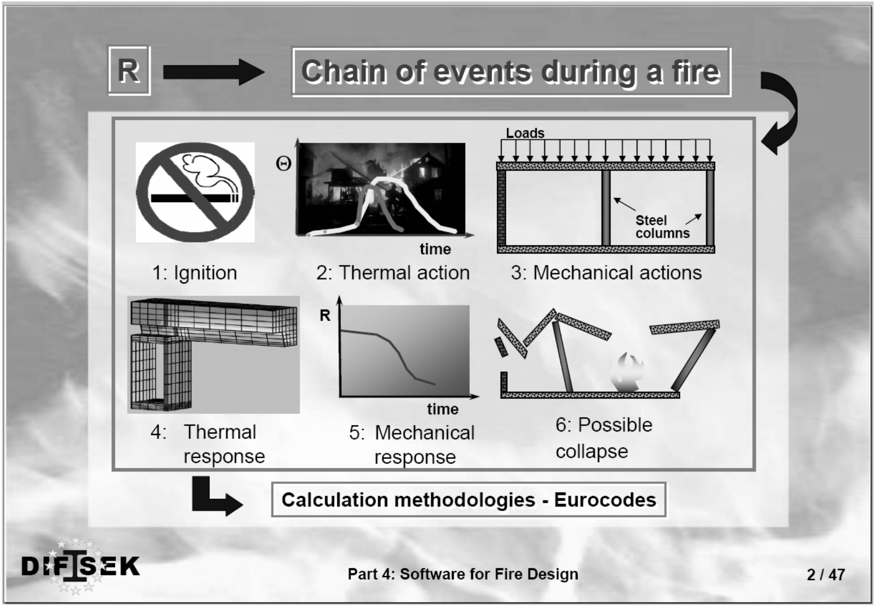

Fig. 11 Example of a PowerPoint page from DIFISEK materials, see [27]

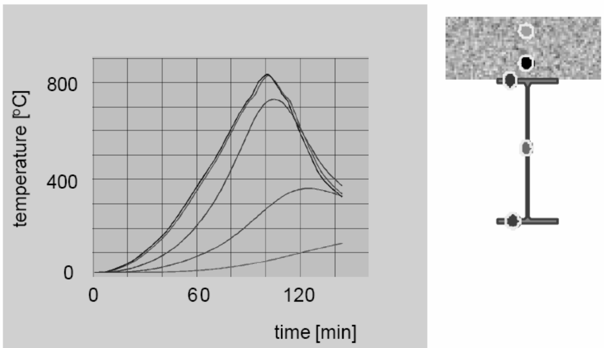

Fig. 3: Thermal response: steel beam/concrete slab (2D)

Fig. 12 Example of a figure form text page in DIFISEK materials, see [27]

Internet tool AccessSteel, URL: access-steel.com, see [28] Fig. 13, represents the last progress in support of design and education of structural steel which been specifically tailored for construction professionals and their clients to offer guidance through project initiation, scheme development, and detailed design. The tool is equipped by robust engine for searching a text through all materials, see Fig. 14. The design materials for single story, multi story and residential buildings are supported by documents related to fire design form the conceptual design to the detail calculations including all the standard references. The site includes over 49 interlinked modules on design in fire engineering, with step-by-step guidance, full supporting information and worked examples, to give a thorough understanding of how the Eurocodes should be used.

Design verification for the Eurocodes covers the four critical steps. Each design activity is described separately by a flow chart, see Fig. 15. A commentary is provided on the effective application of every clause in Eurocode that is referenced. Non-contradictory, complementary information (NCCI) is presented that addresses all the information that the Eurocodes do not cover that is essential for design, see Fig. 16. Worked examples illustrate all the key design stages. The files are available in English, French, German, Spanish, and Czech languages.

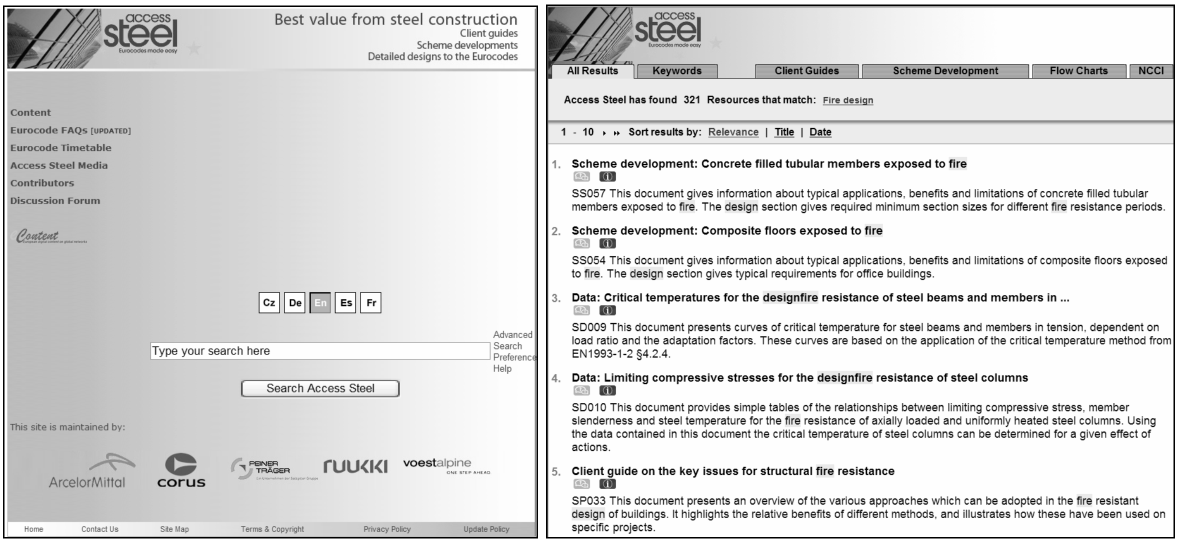

Fig. 14 Fig. 13 Home page of AccessSteel, and result of text, see [28]

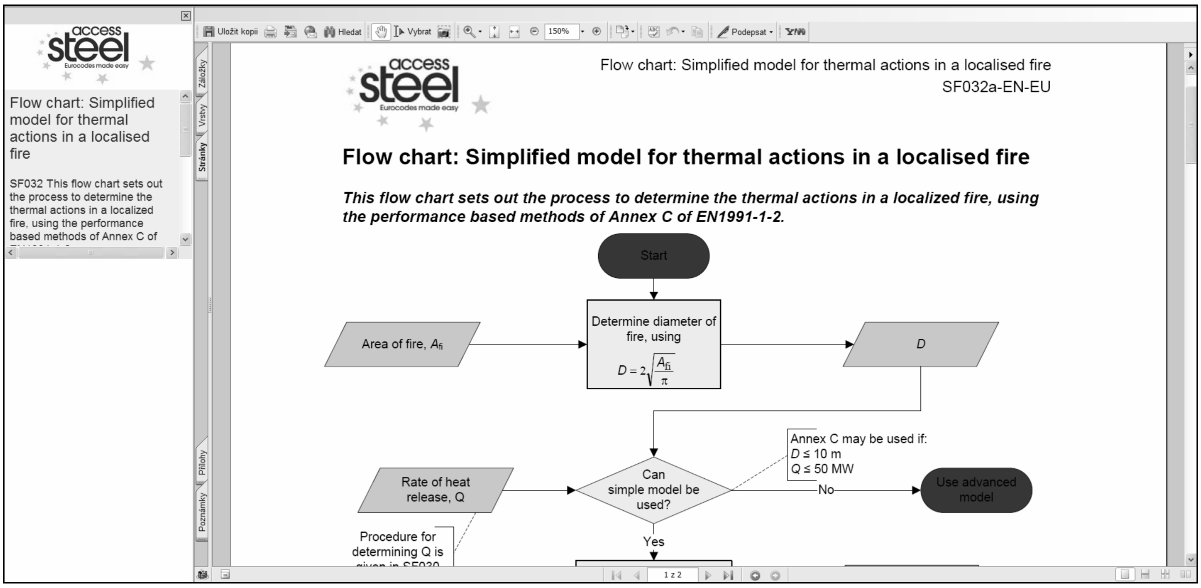

Fig. 15 An educational flow chart for modelling of localised fire in AccessSteel, see [28]

5 FUTURE NEEDS

European knowledge of fire design has reached a mature stage with the development of well calibrated engineering tools for modelling structural behaviour under fire conditions. Four steps of procedure may be identified: modelling fire scenario in the compartment or a local fire, modelling the transfer of heat to the structure, assessment of the mechanical loading under fire conditions, and evaluation of the response of the structure at elevated temperature. Tools for all four separate stages are available for practical application. Merging of these models is also under development in several institutes. The simplest design models are supported by design tables and design charts. More advanced models can deal with natural fire scenarios, refined transfer of heat between the atmosphere and the structure and non-linear large displacement global analyses. The complex models, based on FE modelling of fire scenarios and 3D non-linear behaviour of structures, are ready to be applied for prediction of the structural behaviour under exceptional fire loading.

Nevertheless, applications of these complex models are limited by tests on whole buildings in fire and by confirmation of accurate prediction of internal forces under fire. Further research studies are necessary and indeed some are already been undertaken by various researchers to gain new knowledge so as to develop better tools to help achieve future desired level of fire safety. The fire safety is aim of more running European projects and networks. To the robustness of structures and its connection behaviour during fire is e.g. focussed the work of WG1 of the European network COST C26 Urban Habitat Constructions under Catastrophic Events, see [29].

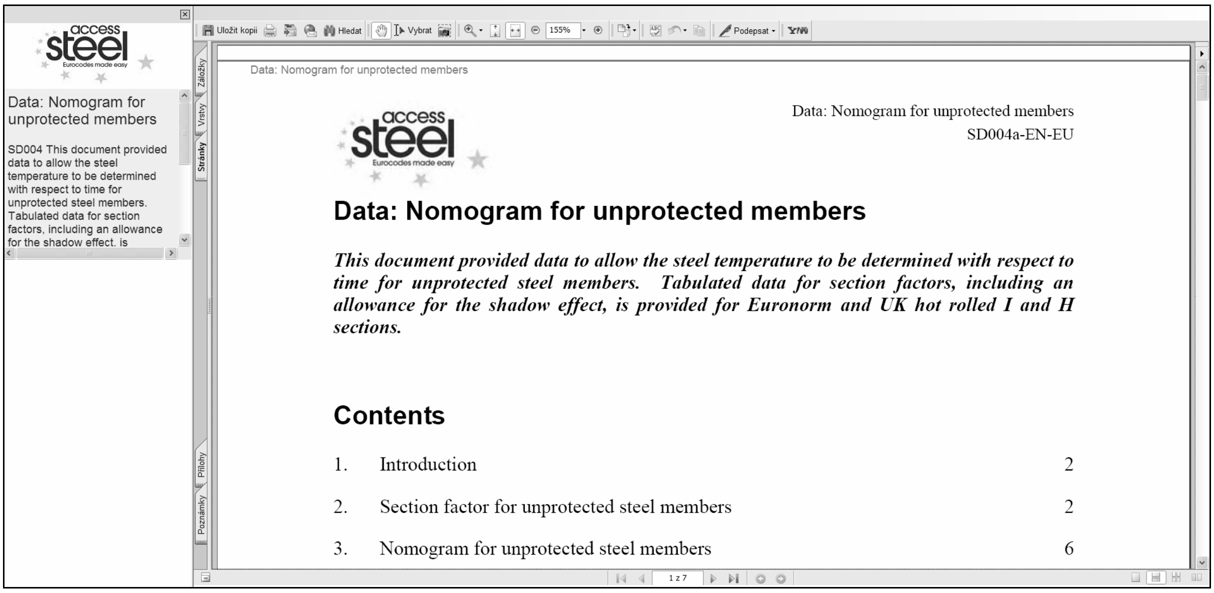

Fig. 16 NCCI for the unprotected member design in AccessSteel, see [28]

Acknowledgement

This outcome has been achieved with the financial support of the Ministry of Education, Youth and Sports of the Czech Republic, project No. 1M0579, within activities of the CIDEAS Research Centre.

References

[1] *The Construction Products Directive*, Council Directive 89/106/EEC, from 21.12. 1989. URL: ec.europa.eu.

[2] Buchanan A. H. *Structural design for fire safety*, John Wiley & Sons 2000, ISBN 0-471-89060-X.

[3] EN 1990-1-2: 2002. Eurocode: *Basis of structural design*, Brussels 2002.

[4] EN 1991-1-2: 2002. Eurocode 1, *Basis of design and actions on structures* – Part 2-2: Actions on structures – Actions on structures exposed to fire, CEN, Brussels 2002.

[5] EN 1993-1-2: 2005. Eurocode 3 – *Design of steel structures* – Part 1-2: General Rules – Structural fire design, CEN, Brussels 2005.

[6] EN 1994-1-2: 2005. Eurocode 4 – *Design of composite steel and concrete structures* - Part 1-2: General rules - Structural fire design, CEN, Brussels 2005.

[7] Schleich J.B., Krupa J., Newman G. & Twilt L. 2001. *Model Code on Fire Engineering*, ECCS No. 111, Brussels, p. 165, ISBN 92-9147-000-65.

[8] Pettersson, O. Magnusson, S.E. & Thor J. Fire engineering design of steel structures, Publication Noº50, Swedish Institute of Steel Construction, Stockholm, 1976.
[9] Drysdale, D. D. *An Introduction to Fire Dynamics,* 2nd ed, West Sussex, England, John Wiley & Sons1999.
[10] Frangi A. & Fontana M. *Versuche zum Tragverhalten von Holz-Beton-verbunddecken bei Raumtemperatur und normbrandbedingungen*, Institut für Baustatik und Konstruktion (IBK), ETHZürich, IBK Bericht Nr. 249, Birkhäuser Verlag Basel, 2000.
[11] Cooke G. M. *Stability of lightweight structural sandwich panels exposed to fire*, Document number 10.1002, New Zealand, 2004.
[12] Franssen, J. M., Schleich, J. B. & Cajot, L.-G. *A simple model for fire resistance of axially-loaded members according to Eurocode 3*; J. Construc. Steel Research, Vol. 35; pp. 49-69, 1995.
[13] Bailey C.G. *Fire design of steel framed structures*, in Steel and Composite Structures. Leiden: Taylor & Francis/Balkema, 2007, Vol. 1, p. 57-65. ISBN 978-0-415-45141-3.
[14] Bailey C.G., Lennon, T. and Moore, D.B. The behaviour of full-scale steel framed buildings subjected to compartment fires, *The Structural Engineer*, Vol. 77, No. 8, April 1999. pp. 15-21.
[15] EN 1363-1:1999. *Fire resistance tests* - Part 1: General requirements, CEN Brussels. 1999.
[16] Newman G.M., Robitson J.T. & Bailey C. G. *Fire safe design, a new approach to multi-storey steel-framed buildings*, SCI, P288, London 2000, ISBN 1 85942 120 2.
[17] Bailey, C. G. & Moore D.B. The structural behaviour of steel frames with composite slabs subject to fire, Part 1: Theory, *The structural Engineer*; Part 2: Design, The structural Engineer, 2000.
[18] Lennon T. *Cardington Fire Tests: Survey of Damage to the Eight Storey Building*, BRE internal Report GD1286/86, 1997.
[19] Wald, F., Simões da Silva, L., Moore, D.B., Lennon, T., Chladná, M., Santiago, A., Beneš, M. & Borges, L. Experimental behaviour of a steel structure under natural fire, *Fire Safety Journal* **,**Volume 41, Issue 7, October 2006, p. 509-522**.**
[20] Wang Y.C. *Steel and Composite Structures, Behaviour and Design for Fire Safety*, Spon Press, London 2002, ISBN 0-415-24436-6.
[21] Moore D.B. & Lennon T. Fire engineering design of steel structures, Progress in Structural Engineering and Materials, 1 (1), pp. 4-9, 1997.
[22] Simões da Silva, L.A.P., Santiago, A. & Vila Real, P. *A component model for the behaviour of steel joint at elevated temperatures*, Journal Constructional Steel Research 57(11), pp. 1169-1195, 2001.
[23] Spyrou S., Davison B., Burgess I. & Plank R. *Component-based studies on the behaviour of steel joints at elevated temperatures*, in Proceedings of Eurosteel 2002 – 3rd European Conference on Steel Structures, edited by Lamas A., Simões da Silva L., s. 1469-1478, Coimbra 2002, ISBN 972-98376-3-5.
[24] Zoetemeijer P. *Summary of the research on bolted beam-to-column connections*, TU-Delft report 26-6-90-2, Delft, 1990.
[25] EN 1993-1-8: 2003. Eurocode 3: *Design of steel structures*, *Part 1.8: Design of joints*, European Standard, CEN, Brussels, 2005.
[26] Wald F., Chlouba J., Sokol Z., Kallerová P.: *Connections with improved fire resistance*, Connections VI, AISC, Chicago, 2008, in printing.
[27] URL: www.difisek.eu
[28] URL: access-steel.com
[29] URL: www.civ.uth.gr/cost-c26