COST C26 – WG1

Datasheet no. 1

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INTRODUCTION TO FIRE DESIGN

Introduction

There are a number of approaches to ensure the safe design of structures under fire conditions. This range is 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, introduces the benefits of using a performance-based approach to fire engineering, and calls attention to European materials, freely available on the Internet, which support such design.

Existing recommendations, models and regulations about structural safety are mostly aimed at ensuring an adequate level of safety for constructions under normal loading conditions. Overall structural safety is dealt with by specifying safety factors, which increase the service loads (or other actions) to high values under which the structure must not collapse. This approach has led to a satisfactory degree of confidence in ensuring the safety margins under all normal load conditions. Greater accuracy in the evaluation of structural safety is possible when a probabilistic or semi-probabilistic approach is followed in the determination of both the design actions and the structural resistance. In this way it is possible to achieve a safe response from structures subjected to random actions. These approaches form the basis of most recent developments in the field of building regulations, and are part of almost all relevant structural codes, including the Eurocodes, in which 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: 21.12.1988). The Directive is applied to construction products as the essential requirement in respect of construction works. In Annex I of the Directive, the essential requirements for mechanical resistance and stability, and for fire safety, are summarised. 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 parts of the structural Eurocodes which deal with fire.

Fire resistance

Fire resistance is commonly used to characterize the performance of elements of structure in fire. It 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 to avoid collapse, to limit the spread of fire and to support other elements. All construction materials progressively lose their ability to support a load when they are heated. If components of any structure are heated sufficiently they may collapse. The consequences of such a collapse vary, depending on how critical the component is in controlling the overall behaviour of the structure. In order to limit the threat posed by fire to people in a building, and to reduce the damage that a fire may inflict, large buildings may be divided into smaller fire compartments using fire-resisting walls and floors. Parts of a fire compartment may be further divided to protect the building from particular hazards within them. The performance of fire separating elements may rely on the ability of their supporting structure to continue to provide support under fire conditions (Buchanan 2000). The criticality of an element is the degree to which its collapse would affect the performance of the structure as a whole. All of the 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 of which the structural element is a part; this is an important part of a fire safety risk analysis.

An amplified definition of the fire resistance of a structure or an element is *its ability to retain for a stated period of time its load-bearing capacity, integrity and insulation, either separately or in combination*. As a consequence of European harmonization, fire resistance is increasingly being expressed in terms of *R* (resistance to collapse, or the ability to maintain load-bearing capacity), *E* (resistance to fire penetration, or the ability to maintain the fire integrity of the element against the penetration of flames and hot gases) and *I* (resistance to the transfer of excessive heat, or the ability to provide insulation to limit excessive temperature rise. The term *element of structure* is used in fire engineering to denote 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. Curtain walls or other forms of cladding which transmit only self weight and wind loads, but do not transmit floor loads, are not regarded as load-bearing, although they may need fire resistance to satisfy the requirement to restrict fire spread between buildings. Load-bearing elements may or may not have a fire-separating function, and conversely fire-separating elements may or may not be load-bearing.

Fire design

Design for fire safety has traditionally followed prescriptive rules, but may now apply fire engineering or performance-based approaches, examples of which are given in documents EN 1990, 2002 and EN 1991-1-2, 2002. A fire engineering approach takes account of fire safety in its entirety, and usually provides a more fundamental and economical solution than the prescriptive approaches. Within the framework of a fire engineering approach, designing a structure involves four stages:

1. Modelling the fire scenario to determine the heat released from the fire and the resulting atmospheric temperatures within the building.
2. Modelling the heat transfer between the atmosphere and the structure. This involves conduction, convection and radiation, which all contribute to the rise in temperature of the structural materials during the fire.

3. Evaluating 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 in fire.
4. Determination of the response of the structure at elevated temperature.

The design recommendations in codes contain simple checks which provide an economic and accessible procedure 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, generally using the finite element method which may predict thermal and structural performance. In fire, the behaviour of a structure is more complex than at ambient temperature, because changes in the material properties and thermal movements cause the structural behaviour to become non-linear and inelastic.

Fire modelling

In standard fire resistance tests the gas temperature follows a predefined time/temperature curve, known in the Eurocodes as the Standard Nominal Fire Curve or alternatively as the ISO 834 Fire Curve. This heating regime is different from that in any real fire. The maximum temperature attained in a real fire, and the rate of temperature increase, depend on a number of factors including 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 (Schleich *et al.,* 2001) to mathematically describe a natural fire. The rate at which heat is released from the available fuel 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 are determined.

Fire models of various degrees of sophistication may be used to obtain design fire scenarios. At the most simplistic level, periods of standard fire resistance are specified in regulations. The next level is to attempt to relate the effect of a real fire to the Standard Fire by using a time-equivalence approach. Ideally, the equivalent time of fire exposure should compare the performance of an element in a natural fire with the known performance of the same element in a fire resistance test. In practice it is often used in a form which is attractive to fire investigators and fire engineers because it allows them to relate the complex behaviour of a real fire itself to a time in the standard fire curve. An equivalent-time equation, expressed as a function of the fire load, ventilation and thermal characteristics of the enclosure, is given in Eurocode 1 Part 1.2.

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 1.2 refers to these as parametric fire curves, and provides equations based on the pioneering research work of Pettersson (1976) to calculate these using the three aforementioned parameters. In its most complex form, fire modelling may involve using computational fluid dynamics (CFD) modelling, (Drysdale 1999) which is computationally very intensive.

Structural response

Structural response and its modelling under fire conditions depend on the structural materials used, as well as the extent of structure modelled, which may be the whole structure, parts of the structure, or individual elements, see (Bailey et al, 1999). Standard fire resistance tests can only provide limited guidance. As far as different materials are concerned (Cooke 2004, Fragi and Fontana 2000), aluminium and steel have high thermal conductivity, and therefore transfer heat rapidly. Timber, masonry, concrete and lightweight concrete have lower conductivity, and therefore better insulation properties. Additional insulation may be economical for aluminium, steel and timber structures. The simplified design models in codes such as the Eurocodes) are mostly based (Fransen *et al.* 1995) on design checks for ambient-temperature design. On the other hand, more advanced models of global analysis using finite elements may be used to deal with the interactions between different structural members and connections, and structural behaviour at large deformations (Bailey, 2007).

Support for design

Various books and software (Buchanan 2000) now exist on fire engineering. Educational material from the RFCS project DIFISEK (DIssemination of structural FIre Safety Engineering Knowledge is also available on the Internet for steel and composite structures. PowerPoint presentations (Figure 1) and lecture notes explain the fire engineering approach, covering the whole spectrum of fire engineering, from the calculation of gas temperatures to the design of structural elements to resist fire. Both prescriptive rules using standard fire curves, as well as the real behaviour of fires, are included. All the DIFISEK materials accord with the Eurocodes. The documents have been translated into German, French, Spanish, Dutch and Finnish, and by the end of 2008 will also be available in Czech, Estonian, Greek, Hungarian, Italian, Lithuanian, Polish, Portuguese, Romanian, Slovenian, and Swedish. In addition, a database dealing with fire design software is provided.

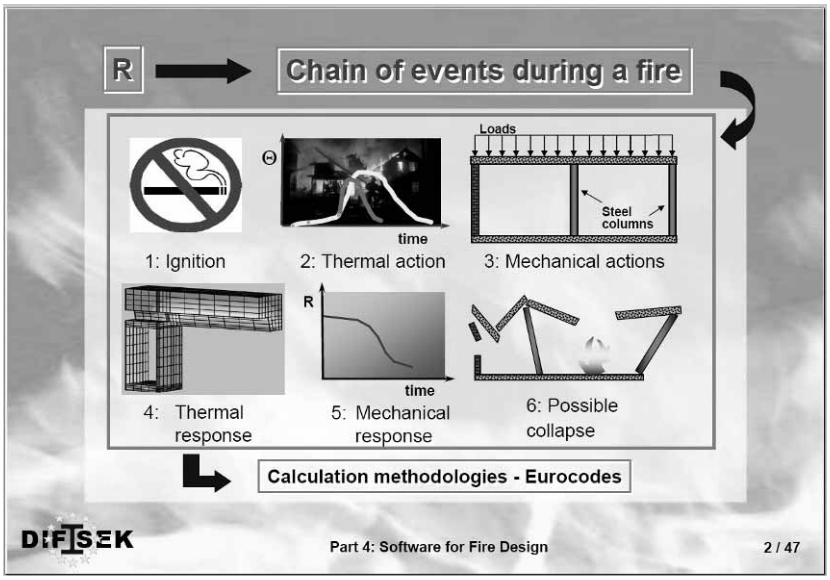

Figure 1. Example of a PowerPoint page from DIFISEK materials.

All language versions, as well as the database and freely available software, are included on a CD-ROM and are also available for free download at www.difisek.eu. 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.

The Internet tool AccessSteel (www.access-steel.com), shown in Figure 2, has been specifically tailored for construction professionals and their clients, to offer guidance through project initiation, scheme development, and detailed design. The tool is equipped with a robust engine (Figure 3) for searching text through all the reference materials in the database. The design materials for single-storey, multi-storey and residential buildings are supported by documents related to fire design, from conceptual design to detailed calculations, including all the standard references. The site contains more than 50 interlinked modules on fire engineering design, including step-by-step guidance, full supporting information and worked examples.

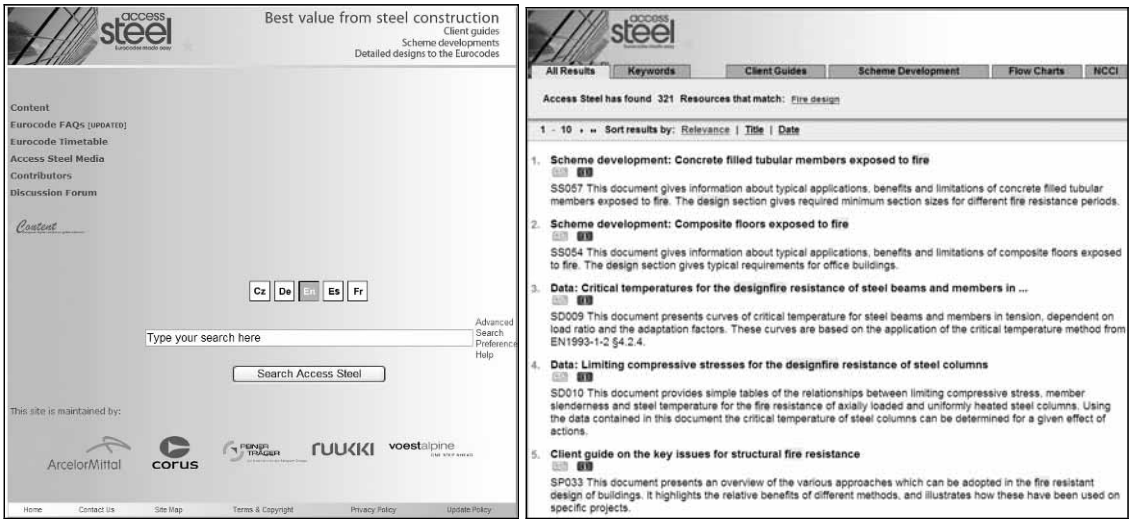

Figure 2. Home page of AccessSteel, and result of text.

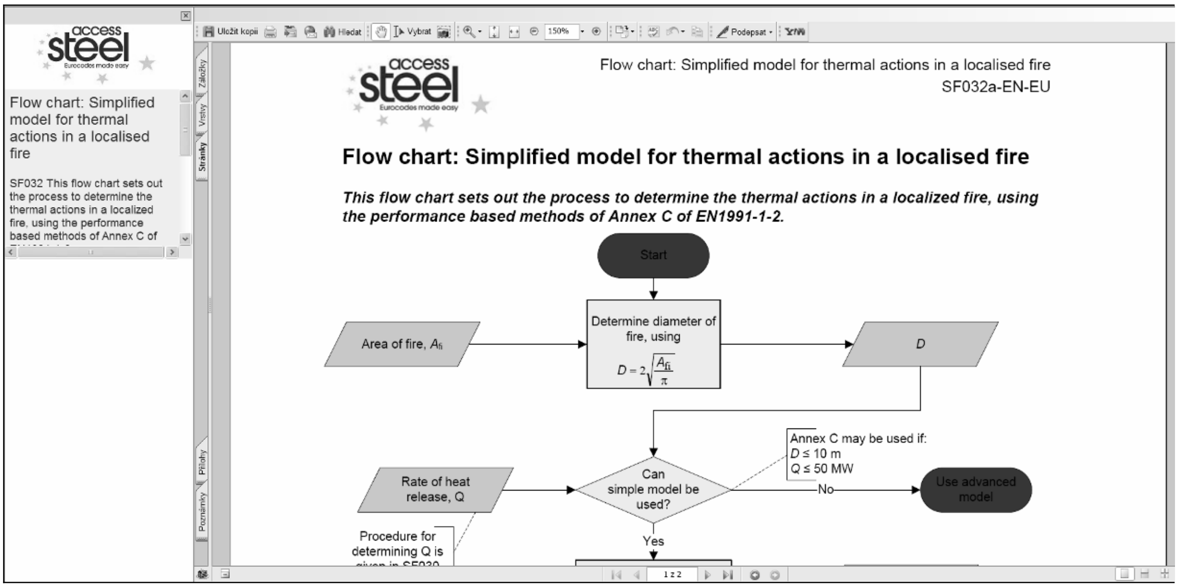

Figure 3. An educational flow chart for modelling of localised fire in AccessSteel.

Design verification for the Eurocodes covers the four critical steps described previously. Each design activity is described separately by a flow chart, as shown in Figure 3. A commentary is provided on the effective application of every Eurocode clause referenced. Non-contradictory, complementary information (NCCI) is presented; this addresses essential information for design that the Eurocodes do not cover. Worked examples illustrate all the key design stages. The information is currently available in the English, French, German, Spanish, and Czech languages.

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COST C26 – WG1

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OVERVIEW OF FIRE DESIGN

Description

- Performance-based fire safety design is an accepted methodology in many countries of Europe for the verification of structural resistance in fire conditions. This calculation procedure takes into account the individual characteristics of the building and the passive and active fire protection methods.
- A realistic understanding of the behaviour of structures in fire can be achieved and the overall safety of the building can be verified by using performance-based fire safety design. Through the more profound understanding of phenomena and a more precise analysis of structures in fire, an equal to or higher safety level than with prescriptive fire design can be obtained.
- Generally more economical designs, compared to the simple prescriptive approaches, whilst still maintaining acceptable levels of life safety. The construction of more innovative and complex buildings which were not possible due to the restrictive nature of the simple prescriptive rules.
- A better understanding of the actual structural behaviour of the building during a possible fire.
- The construction of more robust buildings due to the advanced design approach allowing identification, and strengthening, of any 'weak' links within the structure.
- An increase in the levels of safety offered by the simple prescriptive design approaches, by incorporating advanced structural fire design within a global fire strategy.

Field of application

- Present day structural fire resistance regulations are largely based on the so-called standard fire curve, which has led to very different practices in different European countries. For example, the fire resistance time for a similar building can vary between 60 min in the Netherlands and 120 min in Finland.
- Due to the different uses and other individual characteristics of buildings, fire resistance requirements and design should be based on factors that actually have an influence on the growth and the development of fires, the safety of persons in the specific building as well as the loading conditions of the specific building. Fire safety engineering has been developed into a separate engineering discipline, of which structural fire resistance, which is covered in this technical sheet, forms part of the fire protection system.

Technical information

- Fire resistance is concerned with ensuring that a fire is contained within the compartment of fire origin so that it does not spread. A fire may spread through the compartment of fire origin in three ways: collapse of the compartment structure, excessive temperature rise on the unexposed side to cause further ignition, burning through the compartment. In fire resistance terminology, the ability to prevent fire spread through the above three ways is termed loarbearing capacity, insulation, and integrity. This technical sheet is concerned with structural loadbearing capacity, which is to ensure that the structure has sufficient resistance so that it does not collapse when exposed to fire. Assessment of structural loadbearing capacity maybe considered in either the strength domain or time domain. In the strength domain, the residual strength of the structure under fire attack should not be lower than the applied load in fire. In the time domain, the structure should not collapse before the required fire resistance time is reached.
- The Institution of Structural Engineers in the UK has recently published a guide on structural fire resistance design. A dedicated website (www.structuralfiresafety.com) with free access may be consulted to obtain more detailed guidance, tutorial and reference materials.

Structural aspects

- In general, structural fire resistance calculations may be divided into three steps: evaluation of fire behaviour, calculation of temperatures in structural components and assessment of residual loadbearing capacity.
- Depending on the project requirement, different types of fire exposure may be considered, including the usual standard fire resistance rating, parametric fire curves simulating realistic post-flashover compartment fires or localised fires.
- Having obtained the fire behaviour, the temperatures in different structural members exposed to fire can be obtained by a number of methods, including fire test, tabulated values based on fire tests of validated numerical analysis or through a heat transfer analysis. Technical sheet No. 4 provides detailed method of heat transfer analysis. When carrying out heat transfer analysis, it is important to use appropriate thermal properties of the structural materials and any fire protection materials. The most important thermal properties of materials are thermal conductivity, specific heat and density. It is also important that the appropriate thermal boundary conditions are used. In structural fire engineering calculations, the thermal boundary condition is usually conveniently represented by a heat transfer coefficient, which is divided into a convective heat transfer coefficient and radiant heat transfer coefficient. Under post-flashover fire condition, the radiant heat transfer coefficient is of primary importance. This value directly depends on the fire and structural surface emissivity values.
- Once the structural temperatures are obtained, the residual loadbearing capacity of the structure at elevated temperatures can be calculated. Elevated temperatures have two general effects on a structure: (1) the mechanical properties of structural materials are reduced at high temperatures; (2) thermal elongation (at increasing temperature) and contraction (at reducing temperature) impose additional loading to structural members.
- Mechanical properties of structural materials at elevated temperatures are provided in technical sheet No. 5.
- When checking the loadbearing capacity of a structure at elevated temperatures, the calculations may be carried out at different levels: analysis of members with statically determinate loading condition; part of structure analysis; whole structural analysis. Structural member analysis does not consider interactions between different structural members in fire and methods of checking member capacity at elevated temperatures are

given in technical sheet No. 8. When conducting member analysis and design, it may be necessary to include the contribution of joints. Guidance on joint analysis and design if fire is provided in technical sheet No. 12. Both part of structural analysis and whole structural analysis should consider structural interactions. In particular, when checking structural resistance in fire, the structure is often allowed to develop very large deflections. Such large deflections will necessitate the consideration of structural phenomena that are often ignored in normal structural analysis and design at ambient temperature. For example, beams and slabs are only analysed for their bending moment resistance at ambient temperature so that the in-plane behaviour (catenary action in beams, membrane action in slabs) is not considered. However, such modes of behaviour can have significant impact on structural fire resistance. Technical sheet No. 6 provides further guidance.

- In addition to assessment of structural loadbearing capacity under the expected fire condition, it is also important to make provision for the "unexpected" consequence of fire attack (exceptional fire loading). This is the same as checking for structural robustness. The difference between normal structural robustness (control of progressive collapse) and design for exceptional fire loading is that the additional requirement of fire spread through compartments should be considered. Technical sheet No. 15 provides some provisional guidance on design for structural robustness and fire integrity.
- Structural fire safety design interacts with both the structural engineering profession and the fire engineering profession. Therefore, when carrying out structural fire safety design and analysis, it is important that the designer interacts with the two closely related professions as well as the client, the architect and the fire and building authorities as early as possible so that important decisions such as the safety level to be achieved, the choice of design fire and the objectives of structural fire engineering design are clearly agreed.
- The choice of the critical design fires for the designed building is an important phase in performance-based fire design. The number of possible fire scenarios is of course very large, but only a part of them can be considered critical and require further analysis. The characteristics and number of design fires depends on e.g. the geometry of the compartment, the use of the building, the fire load etc. The degree of criticality and probability of occurrence of different fire scenarios should be determined. It is also important to remember to carry out sensitivity analyses on different factors. Fundamentally the choice of design fires is the job of the building administration authorities and they should be discussed in the Fire Engineering Briefing at the start of the project.

Guidelines

- The Institution of Structural Engineers in the UK has published a design guide to fire safety engineering of structures. A list of contents of this guide is provided on Fig. 1 for reference. Similar guidelines may be found in most European countries see Technical sheet No. 3.

Figure 1. List of contents ISE (2007).

Definitions

- *Design fire*: a specified fire temperature development assumed for structural design purposes.
- *Fire resistance*: the ability of a structure, a part of a structure or a member to fulfil its required functions (load bearing function and/or separating function), for a specified load level, for a specified fire exposure and for a specified period of time.
- *Fire scenario*: a qualitative description of the course of a fire with time identifying key events that characterise the fire and differentiate it from other possible fires. It typically defines the ignition and fire growth process, the fully developed stage, decay stage together with the building environment and systems that will impact on the course of the fire.
- *Indirect fire actions*: internal forces and moments caused by thermal expansion.
- *Load bearing function*: the ability of a structure or a member to sustain specified actions during the relevant fire, according to defined criteria.
- *Nominal fire*: conventional design fire, adopted for classification or verification of fire resistance, e.g. the standard temperature-time curve.
- *Separating function*: the ability of a separating element to prevent fire spread (e.g. by passage of flames or hot gases) or ignition beyond the exposed surface during the relevant fire.
- *Standard temperature-time curve*: a nominal curve for representing a model of a fully developed fire in a compartment.

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Datasheet no. 3

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FIRE DESIGN IN EUROPE

Description

- European countries are now starting to apply the EN standards for buildings.
- The EN standards include principal and application rules for fire resistance design of buildings allowing also performance based fire design.
- What is the state of art for applying the EN standards for fire resistance design and especially performance based design?
- For which kind of applications there exist experiences to use the performance based design in fire?
- These were the main questions to which were looked for answers by questions sent to the members of the COST C26.

Field of Application

- This datasheet is concerned with the summary of the query done during spring 2008.
- The questions were as follows
  - National fire code includes performance based fire design?
  - EN 1991-1-2 - EN 1996-1-2, EN 1999-1-2 used since?
  - Status of application of Annexes A-G of EN1991-1-2 in your country?
  - Application of EN 1991-1-2, clause 4.3.1(2) (load factors in fire)?
  - Performance based fire design used in real projects? If yes, please give examples of tasks and projects.
  - When used, approval of design needed by third part?
  - Which authorities (Building, Fire) give the official acceptance and in which state of design?
  - Is there specific competence/qualification criteria for the fire engineering designers?
  - Is there enough education for the fire engineering and structural engineering?
  - Requirements for the contents of the documentation on fire design?
  - Problems in the applications of performance based fire design?
  - Programs used for fire simulation?
  - Programs used for evacuation simulation?
  - Programs used for resistance checks?
  - Other items?
  - List of projects, research and development activities included?
- Total amount of 12 answers were got from 10 countries. Three different answers were got from UK.

Technical information

Standards

- National fire code includes performance based design in Czech Republic, UK, Finland, Hungary, and Italy. It is possible in Belgium if a derogation to the Fire Regulation is agreed on by decision of the Minister of Interior. In France it is possible to apply it partially for fire resistance and smoke propagation.
- In UK they have used the performance based fire design for the longest time.
- The following table includes the years when in different countries have started or are planning to start to use fire Eurocodes.
- In the table can be seen the countries which provided the answers to the query.
- The years 2008 and over mean typically: expected to be published.

Table 1. Use of fire Eurocodes in ten European countries

	Belgium	Czech	England and Wales	Finland	France	Hungary	Italy	Poland	Portugal	Romania	UK (Arup)	UK
EN 1991-1-2 (loads)	2008	2006	2002	2007		2005	2008	2006	2008	2007	2002	2002
EN 1992-1-2 (concrete)	2008	2006	2004	2007		2005	2008	2008	2008	2008	2004	2004
EN 1993-1-2 (steel)	2008	2006	2005	2007		2005	2008	2007	2008	2007	2005	2005
EN 1994-1-2 (composite)	2008	2006	2005	2007		2005	2008	2008	2008	2007	2005	2005
EN 1995-1-2 (wood)	2008	2006	2004	2007		2005	2008	2008	2008	2007	2004	2004
EN 1996-1-2 (masonry)	2008	2006	2005	2008		2005	2008	2009	2008	2008	2005	2005
EN 1999-1-2 (aluminium)	2008	2008	2007	2008		2007		2010	2008	2008	2007	2007

- It can be seen, that only France is waiting for a decree shifting from ENV to EN.
- It can be seen, also, that in UK they have been used the final Eurocodes for the longest time.
- The next table includes the use of Annexes of EN 1991-1-2 in different countries.

Table 2. Use of EN 1991-1-2 Annexes in ten European countries

EN 1991-1-2	Belgium	Czech	Finland	France	Hungary
Annex A allowed?	For first approximation	Yes, informative	Yes	Only in pre-design	Yes
Annex B allowed?	Yes, informative	Yes, informative	Yes	Yes	Yes
Annex C allowed?	Yes, normative	Yes, informative	Yes	Informative, need to have peer review	Yes
Annex D allowed?	Yes, normative	Yes, informative	Yes	Informative, need to have peer review	Yes
Annex E allowed?	Yes, informative	Yes, informative	E.4 yes, others: no	No	Yes
Annex F allowed?	No	Yes, informative	No	No	Yes
Annex G allowed?	Yes, normative	Yes, informative	Yes	Informative	Yes
EN 1991-1-2	Italy	Poland	Portugal	Romania	UK
Annex A allowed?	Yes, informative	Yes	Informative	Yes	Yes with PD 6688-1-2:2007
Annex B allowed?	Yes, informative	Yes	Informative	Yes	Yes with PD 6688-1-2:2007
Annex C allowed?	Yes, informative	Yes	Informative	Yes	No, use PD 6688-1-2:2007 as replacement
Annex D allowed?	Yes, informative	Yes	Informative	Yes	Yes
Annex E allowed?	Yes, informative	Yes	Inf. with changes to E.1	Yes	No, use PD 6688-1-2:2007 as replacement
Annex F allowed?	Yes, informative	Yes	No	Yes	No, use PD 6688-1-2:2007 as replacement
Annex G allowed?	Yes, informative	Yes	Informative	Yes	Yes

- Five countries do not allow the use of the Annex F dealing with equivalent time of fire exposure.
- There are limitations to use of Annex E (fire load densities), and one for Annex C (localised fire).
- In two countries the Annex A (parametric temperature-time curves) is allowed to be used only in the preliminary design stage.
- In France the use of Annexes C and D need to have a peer review of the assessment report according to clause 15 of decree 22 March 2004.
- In these 10 countries the Annexes B (thermal actions for external members – simplified calculation method) and G (configuration factor) are accepted as they are.
- The detailed question dealt with the national annex for EN 1991-1-2, clause 4.3.1(2) dealing

with the representative values of variable actions in fire. Both combination factors ψ_1 and ψ_2 are used.

- In Italy and Romania the factor ψ_2 is used. In France and Portugal (and Spain, Estonia, Slovenia*, information got from DIFISEK+ project) the factor ψ_1 is used. In UK the factor ψ_1 is used for EQU cases and the factor ψ_2 is used for STR cases. In Belgium (and Netherlands, Luxembourg*) the factor ψ_2 is used, but for wind the factor ψ_1 is used. In Czech the factor ψ_2 is used, but for wind and snow the factor ψ_1 should be used. In Finland the factor ψ_2 is used for live loads but the factor ψ_1 for wind, snow and ice actions.
- It can be seen many different ways in Europe to define the actions in fire.

Use of performance based fire design

- Performance based fire design has been used in real projects in 8 countries of 10 which answered to the query.

- Performance based fire design has been used to the following tasks:
  - fire resistance of structures,
  - evacuation calculations,
  - smoke control,
  - risk analysis,
  - optimization the fire protection requirements in structures,
  - studies of local fires,
  - studies of external flames,
  - studies of equivalent times of exposures,
  - demonstration of adequate fire fighting provisions,
  - demonstration of extended travel distances,
  - demonstration of an acceptable standard of safety in complex buildings with large numbers of people and/or large open spaces.

- It can be seen many kinds of applications where performance based fire design has been applied in Europe.
- Typically performance based fire design has been used in large projects, but increasingly also in other projects.

- Typical projects are such as
  - shopping centres,
  - office buildings,
  - airports,
  - hospitals,
  - residential buildings,
  - stadiums,
  - music halls,
  - underground facilities,
  - industrial buildings,

- historical buildings,
- high rise buildings,
- car parks,
- libraries,
- churches,
- monumental buildings,
- warehouses.

- One interesting project was cruiser ships.
- It can be seen wide and interesting variety of the projects where performance based fire design has been used.

- The example projects are such as
  - Fire resistance, evacuation: Terminal Mošnov, (Czech)
  - Fire resistance: Storage hall Mnichovo Hradiště, (Czech)
  - All the major shopping centres in the Helsinki region as well as in other parts of Finland,
  - Office buildings (e.g. Nokia headquarters, the Sanomatalo in Helsinki, Finland),
  - Bucharest Tower Center
  - Evacuation calculations: East London Line Stations, BSF School, Birmingham,
  - Smoke flow analysis: Feature Tower at the ADNEC site, Abu Dhabi, Larnaka Airport, Cyprus,
  - Structural fire protection calculations: River Quarter II Residential Building, Sunderland, Sports Pavilion, Bradford Grammar School,
  - Thermal radiation analyses: Bold Lane Development, Derby, Finzel's Reach, Residential Re-development, Bristol,
  - Heathrow Terminal 5,
  - New Air Traffic Control Tower at London Heathrow Airport,
  - Redevelopment of the historic Spitalfields Market in Central London,
  - New research building for Queen Marys, University of London,
  - Project Emma, HMS Nelson, Portsmouth,
  - Alnwick Castle,
  - Lloyds Registry of Shipping,
  - School of Slavonic and East European Studies, University College London,
  - Queen Mary's New School for Dentistry and Medicine.

- When using performance based fire design then the approval of design by the third part is either required or not. The following table includes the situation in different countries.

Table 3. Need of approval of third part for performance based fire design

Belgium	Czech Republic	England and Wales	Finland	France
No. Approval requested by derogation to Fire Regulation	NO	This varies from project to project and is typically influenced by the ability/experience of the authority having jurisdiction and the complexity of the approach being adopted.	Basically no, depends on the designer	- Firstly agreement is necessary by local authorities on the design fire scenarios. - Secondly, when dealing with fire resistance a peer review (by one of the two agreed laboratories) is necessary. For smoke control an official recognition by the Ministry of Interior is necessary (3 are currently recognised) and no further peer review is necessary.

Hungary	Italy	Poland	Portugal	Romania	UK ARUP	UK
	Yes	Yes, except for small buildings	Typically yes	Typically yes. In fact, fire verification is a problem of structural resistance and therefore must be verified by third part.	Typically, yes, when the structure or building departs from a regular code compliant design.	Normally no, however CFD analysis may require approval by third party.

- The authorities which give the official acceptance for the fire designs are given in the following table.

Table 4. Which authorities, Building or Fire, give the official acceptance and in which stage of design?

Belgium	Czech Republic	England and Wales	Finland	France	Hungary
Can be the local fire brigade, or the Commission of Derogations of the Ministry of Interior	Fire	The Building Control is responsible for enforcing the building regulations. In some buildings they are required to consult with the Fire Brigade, but in others they are not. In either case, they are not obliged to follow the guidance of the Fire Brigade.	Finally Building, statement needed from Fire	Both: at the end of the assessment.	On case-by-case basis by the local fire protection authority The Fire Department gives a general official acceptance for the building permit documentation; the construction work plans should be consulted with the Fire Department; The Fire Department gives official acceptance for the complete construction work plans of the active systems (fire detection and alarm system, sprinkler etc)

Italy	Poland	Portugal	Romania	UK ARUP FIRE LONDON	UK
Fire	Fire authorities	Both	Both	The Building authority. Typically at RIBA Stage D – Scheme Design.	Normally building control officers or approved inspectors. Normally in Stage E.

- Typically no specific qualifications are needed for the fire designer, but in many countries they may require some in the future.
- E.g. in Finland exists certificates for fire safety and structural engineers and in practise it is frequently required by the authorities, that those are acting in the projects where performance based fire design is used.
- Requirements of the documentation on fire design are given in different countries in the following table.

Table 5. Requirements of documentation

Belgium	Czech Republic	Finland	France	Hungary	Italy	Poland	Portugal	Romania	UK
No	Standard ČSN 73 08 18 Fire protection of buildings - Person/surface rate in buildings, ČNI Praha 1974	Yes, Section 1.3.2 of the Finnish National Building code Part E1	no really, but there is an agreement between the two major laboratories (CSTB and Efectis) for such a content	Yes	Yes, in particular in the Decree of the Minister of Interiors of 22 May 2007 and Section 3.6.1 of the new Italian Technical Code for Constructions (2008)	Yes	No	No	No. It is up to the engineer provide sufficient evidence to support their proposals. There are guidance documents such as BS7974 that suggest structures and contents for the documentation, but there is no obligation to follow these documents.

- The main problems when applying performance based fire design are: lack of experience and confidence of the authorities, how to define design fires and parameters in some cases, lack of design tools.

Software

- The programs used for performance based design are given in the next table.

Table 6. Programs used for performance based fire design

Country	Belgium	Czech Republic	England and Wales	Finland	France	Hungary	Italy	Poland	Portugal	Romania	UK ARUP FIRE LONDON	UK
Programs used for fire simulation	OZone	ParamTTcurve, OZONE, Fluent,	In-house software, CFX, FDS, TASEF, Branzfire, FPETool, CFAST.	FDS, OZONE	FDS, OZONE, CFAST,	Easy Flow, NIST Fire Dynamics Simulator, but extremely seldom, only for expert's opinions and research works	FDS, OZONE, CFAST	Commercial software and individual developed for specific problems	ANSYS CFX 5.7.1, OZONE	Up to this date, all fire design was made considering standard ISO fire.	FDS, FPE TOOL, CFAST	FDS and in-house programs
Programs used for evacuation simulation	No	No, simple standard calculations ČSN 73 08 18	BuildingExodus, STEPS	FDS + EVAC	Not very often, FDS + EVAC, Exodus	No programmes used	FDS + EVAC	Commercial software and individual developed for specific problems		No	Legion, STEPS	EXODUS
Programs used for resistance checks	SAFIR	FireResistance, ANSYS, ConTemp, SAFIR	Vulcan, Robot.	COMCOL, WINRAMI, COMSLAB, ROBOT, ESA, ANSYS, ABAQUS	ANSYS, Safir, Castem	No programmes used	SAFIR, STRAUSS, ANSYS, ADINA, ABAQUS	Mostly commercial software (ABAQUS, NASTRAN, LS-DYNA)	SAFIR, ANSYS	SAFIR	ABAQUS, ANSYS, TSLAB, Bailey Spreadsheet Method, SAFIR, STRAND-7	SAFIR, Vulcan and in-house programs

- It can be seen, that much software is available and new ones are under development all the time.

Education

- The education for fire engineering seems to be at very low level in 10 countries involved.
- Often they are required to recruit from other engineering disciplines and re-skill good quality fire engineers.
- In Czech Technical University in Prague is in each February educational happening.
- In France a specific national research project is working on educational subjects.
- In Poland new university education degree system introduces new fire engineering courses to fill the gap.
- In many universities exists optional courses in the main civil engineering faculties.

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Datasheet no. 4

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HEAT TRANSFER ANALYSIS

Description

- Prediction of fire resistance of a structure follows three general steps: evaluation of fire exposure condition (fire dynamics), calculation of structural temperatures under fire exposure (heat transfer analysis), determination of structural resistance at elevated temperatures (structural analysis). Heat transfer analysis links fire dynamics with structural analysis.
- Structural temperatures may be obtained by many means, including fire tests, look up tables or graphs or calculations. Look up tables or graphs are based on limited fire tests or calculations. Their application ranges are limited. This technical sheet gives information on heat transfer analysis by calculations.

Field of Application

- There are three modes of heat transfer: conduction, convection and radiation.
- In structural fire engineering, the heat transfer problem is much simplified. Here, conduction refers to heat transfer within the solid of structural or other constructional elements. Convection and radiation define the boundary condition to conductive heat transfer in the solid elements.
- Depending on the complexity of a problem, heat transfer analysis may be performed using simple analytical equations, or numerical methods. This technical sheet will give the relevant heat conduction equations and associated boundary conditions, and some analytical results.

Technical information

- Depending on the assumption of temperature distribution in a structural element, numerical heat transfer analysis may be 1, 2 or 3 dimensional. An example of 1-D heat transfer occurs in walls or floors exposed to fire attack from one side and ambient temperature on the other side; a composite steel/concrete structure beam/column may be assumed to have the same temperature along the longitudinal direction so heat transfer in this structural member may be assumed to be 2-dimensional; heat transfer in a composite joint represents an example of the general case of 3-dimensional heat transfer.
- The basic Fourier's heat conduction equation in 1-dimension is:

$$\dot{Q} = -k\frac{\partial T}{\partial x}$$

where x is the coordinate; T is temperature; k is the thermal conductivity of the material and $\dot{Q}$ the amount of heat (rate of energy) conducted in the x-direction. The negative sign indicates that heat is conducted from high temperature to low temperature.

- Consider an infinitesimal volume dx.dy.dz in Cartesian coordinates as shown in Figure 1, the general 3-dimensional heat conduction equation is:

$$k_x \frac{\partial^2 T}{\partial x^2} + k_y \frac{\partial^2 T}{\partial^2 y} + k_z \frac{\partial^2 T}{\partial^2 z} + q_g - \rho C_\rho \frac{dT}{dt} = 0$$

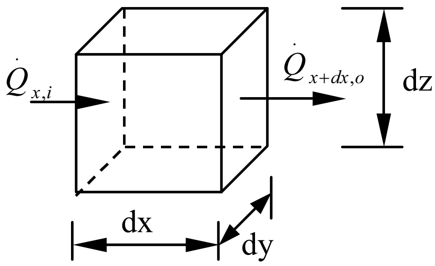

Figure1. Heat conduction in an infinitesimal volume dx.dy.dz

where k_x, k_y and k_z are thermal conductivities of the material in the x-, y- and z- directions respectively; if the material is isotropic, $k_x = k_y = k_z$; q_g is the internal heat generation which should be zero for non-combustible materials; ρ is density of the material and C_p is specific heat of the material; T is temperature and t time.

- When performing heat transfer calculations, it is important to use appropriate thermal properties of materials. These thermal properties are generally temperature dependent.
- Thermal property data of steel and concrete may be obtained from EN 1992-1-2 (for concrete), EN 1993-1-2 (for steel) or EN 1994-1-2 (for composite construction). One chapter of the SFPE handbook on fire protection engineering (SFPE 2002) gives some information on thermal properties of a variety of construction materials.
- Table 1 gives indicative values of thermal properties of a few types of fire protection material (Wang, 2002). More accurate temperature dependent thermal properties of fire protection materials may be obtained from an assessment of fire tests following the procedure in Eurocode 13381 Part 4. In this procedure, steel sections of different sizes protected by the fire protection material of different thicknesses are tested under the standard fire exposure and their temperatures measured. By assuming a nominal density and specific heat for the fire protection material, the temperature dependent thermal conductivity of the fire protection material may be obtained from the measured steel temperatures. It is important to recognise that this procedure is suitable only to fire protection materials whose thermal properties are temperature dependent only. It is not suitable to use this method for intumescent coating. This is because intumescent coating is a reactive fire protection material whose performance depends not only on its temperature, but also on the fire exposure condition, its thickness and the protected steel plate thickness (Yuan and Wang, 2008). At this stage, advice from intumescent coating manufacturers should be sought on the thermal properties of their intumescent coating product.
-

Table 1. Thermal properties of fire protection

Material	Density (kg/m^3)	Specific heat (J/kg K)	Thermal conductivity (W/mK)	Moisture content (% by wt.)
Sprayed mineral fibre	250-350	1050	0.1	1.0
Vermiulite slabs	300	1200	0.15	7.0
Vermiculite/gypsum slabs	800	1200	0.15	15.0
Gypsum plaster	800	1700	0.20	20.0
Mineral fibre sheets	500	1500	0.25	2.0
Aerated concrete	600	1200	0.30	2.5
Lightweight concrete	600	1200	0.80	2.5
Normal weight concrete	2200	1200	1.70	1.5

- There are three types of boundary conditions to the conductive heat transfer analysis problem: (1) the temperature is given; (2) the heat flux to the boundary is known; (3) the boundary exchanges heat with another medium of given temperature. In structural fire engineering, the boundary condition is primarily associated with type 3. Here, the structural surface (boundary) is exchanging heat with the fire exposure or ambient temperature.
- For type 3 boundary condition, the boundary condition may be expressed as:

  $$\dot{Q} = \alpha(T_f - T_s)$$

  where T_f is the known medium (fire or ambient temperature air) temperature, T_s the unknown surface (boundary) temperature and $\dot{Q}$ is the heat flux from the medium to the boundary; α is the heat transfer coefficient.
- The heat transfer coefficient may consist of two parts: the convective heat transfer coefficient α_c and the radiant heat transfer coefficient α_r. If the boundary is in contact with the medium, both convective and radiant heat transfer coefficients should be included. If the boundary is not in contact with the medium, $\alpha_c = 0$.
- In structural fire engineering, the so-called post-flashover fires are the primary concern of design. Under this circumstance, the fire temperature is very high and radiant heat transfer dominates the boundary condition. It is acceptable to make gross assumptions on the convective heat transfer coefficient. In EN 1991-1-2, $\alpha_c = 25$ W/m^2 on the exposed side in case of nominal standard fire (35 W/m^2 for natural fires) and $\alpha_c = 9$ W/m^2 on the unexposed side (in contact with the ambient temperature air).
- In structural fire engineering, graybody surface is assumed for radiant heat transfer analysis and each graybody surface has an emissivity which is temperature and wave length independent. In general, radiant heat transfer is complex even with the aid of graybody surface assumption. The network method for radiant heat transfer between graybody surfaces may be used to solve the problem (Karlsson and Quintiere 2000). For the problem of radiant heat transfer in structural fire engineering in which the structural element surface is in contact with the fire, the radiant heat transfer problem is greatly simplified and the radiant heat transfer coefficient can be obtained from:

  $$\alpha_r = \varepsilon_r \sigma \left(T_f^2 + T_s^2\right)\left(T_f + T_s\right)$$

  where σ (=5.68x10^{-8} W/(m^2.K^4)) is the Stefan-Boltmann constant and ε_r is the resultant emissivity, which is calculated from:

  $$\varepsilon_r = \frac{1}{1/\varepsilon_f + 1/\varepsilon_s - 1}$$

  in which ε_f is the emissivity of the fire (for the ambient temperature air, $\varepsilon_f = 1$) and ε_s is the emissivity of the surface of the structural element. When calculating the radiant heat transfer coefficient, it is important to remember to use absolute temperature, which is marked as T in K (not θ, which indicates temperature in ^{o}C).
- For fire exposure to external structural members, the structural member may not be in direct contact with the fire. The radiant heat transfer problem becomes complex. To help solve the problem, the configuration (or view) factor should be included. The view factor expresses the portion of radiant heat leaving the emitter and incident on the receiver. Assuming the emitter is a surface (A_1) and the receiver is a point with an infinitesimal area dA_2, the radiant heat incident on dA_2 is:

$$\dot{Q}_{A_1 \to dA_2} = \Phi E_1 . dA_2$$

where E_1 is the radiant heat per unit area of the emitter surface A_1. Φ is the configuration factor, calculated using the following equation:

$$\Phi = \int_{A1} \frac{\cos\phi_1 \cos\phi_2}{\pi r^2} dA_1$$

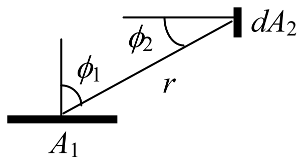

Figure 2. Geometrical dimensions for radiant heat transfer

where the various geometrical dimensions are defined in the Figure 2.

- Radiant heat transfer is the basis for calculating building separation distance.
- Numerical heat transfer analysis may be carried out using a number of specialist (e.g. SAFIR, TEMPCALC, FIRES-T3) or general finite element packages (e.g. ABAQUS, ANSYS, DIANA).
- Approximate analytical solutions are available for a number of simplified situations.
- For a steel structural section without external fire protection exposed to fire around the section, if the steel section is not too thick (< 60 mm), it is acceptable that the steel section has the same temperature and this temperature may be calculated using the following equation:

$$\Delta T_s = \frac{h_r + h_c}{\rho C_\rho} \frac{A_s}{V} \left(T_{f,t+\Delta t} - T_{s,t} \right) \Delta t$$

where ΔT_s is the increment in steel temperature during the time interval Δt (≤ 5 s); A_s is the perimeter area of the exposed steel surface and V is the volume of steel being heated. For a planar steel cross-section, the ratio of A_s/V can be replaced by H_p/A in which H_p is the perimeter length of the exposued steel cross-section and A the steel cross-sectional area.

- For a steel structural section with external fire protection exposed to fire, the steel temperature may be calculated using the following equation:

$$\Delta T_s = \frac{\left(T_f - T_s\right) A_s / V}{\left(d_p / k_p\right) C_s \rho_s \left(1 + \frac{\phi}{3}\right)} \Delta t - \left(e^{\phi/10} - 1\right) \Delta T_f \text{, with } \phi = \frac{C_p \rho_p}{C_s \rho_s} d_p A_s / V$$

where the subscripts "p" and "s" refer to the fire protection material and steel respectively. $\Delta t \leq 30$ s.

- A_s/V (or H_p/A) is commonly referred to as the section factor. Steel sections with a higher section factor value (thin steel) will increase in temperature at a faster rate and vice versa. Section factors for steel sections can be easily calculated. For steel connections, the above equations can also be used. However, it is important that appropriate section factors are used (Ding and Wang 2007, Dai et al 2008).
- The international units of different parameters involved in heat transfer analysis are: length (m), time (s), temperature (K), energy (J), heat ($J.s^{-1}$ = W), density ($kg.m^{-3}$), specific heat ($J.kg^{-1}.K^{-1}$), thermal conductivity ($W.m^{-1}.K^{-1}$), heat transfer coefficient ($W.m^{-2}.K^{-1}$).

Structural aspects

- Heat transfer analysis is the link between fire exposure to structures and calculation of structural behaviour and capacity at elevated temperatures. Therefore, the accuracy of calculations for fire resistance of a structure depends not only on the structural calculations, but also on heat transfer analysis. As an example, consider a steel section with a section factor of 150 m^{-1} which is protected by 20 mm mineral fibre sheets. Assume the thermal properties of mineral fibre sheets are as in the above table but the thermal conductivity ranges within 0.2 - 0.3 $W.m^{-1}.K^{-1}$. Under the standard fire exposure for 60 min, the steel temperature can be calculated using the equation above for protected steelwork. The results are 525°C, 598°C and 656°C respectively for thermal conductivity values of 0.2, 0.25 and 0.3 $W.m^{-1}.K^{-1}$ respectively. Assuming these temperatures are limiting temperatures of a short steel column, then the limiting load ratios according to EN 1993-1-2 are 0.65, 0.45 and 0.3 respectively. This demonstrates that when this is uncertainty on thermal properties of materials, it is important to carry out sensitivity analysis and make appropriate engineering judgement.

- Under the standard fire exposure, the fire temperature is monotonically increasing and the structural temperatures are also monotonically increasing. Therefore, the heat transfer calculations can be terminated at the design fire rating when the maximum structural temperatures (hence minimum structural capacity) are reached. Under more realistic fire conditions, the fire has a cooling down phase. The maximum structural temperatures are often reached after the fire has reached its peak temperature, see (Wald et al, 2006) and Figure 3 for example. Therefore, the structural capacity may still decrease after the fire has reached its peak temperature. It is important that heat transfer and structural engineering calculations are continued until the minimum structural capacity is reached.
- For simplicity, heat transfer calculation is usually decoupled from structural calculations, implying that there is no effect of structural behaviour on heat transfer in fire. This assumption is acceptable in most cases. However, in situations where large distortions of structural members drastically alter structural geometries (e.g. severely deformed joints) or boundary conditions are changed (e.g. large cracks in concrete slabs, spalling), an interactive heat transfer/structural analysis may have to be performed. At present, this remains as a challenging research topic and engineering judgement should be exercised in fire engineering design.

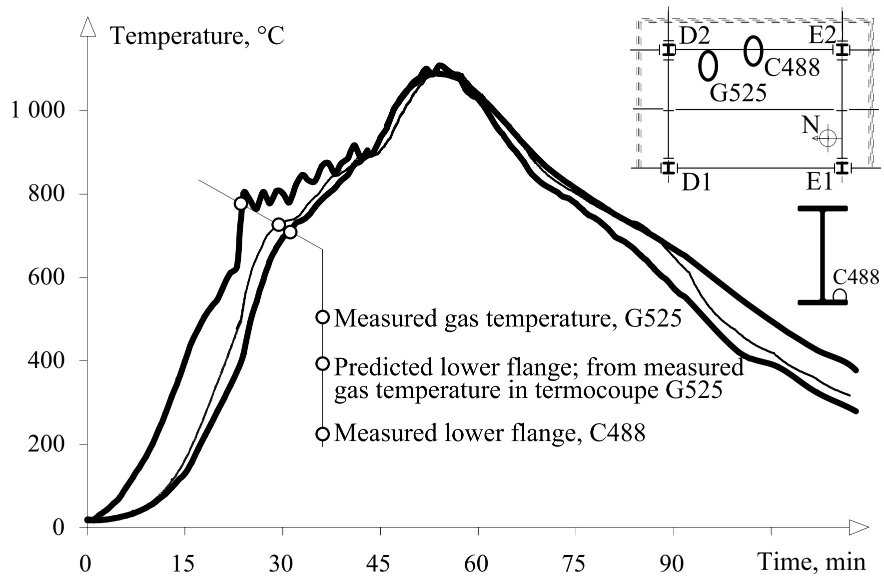

Figure 3. Prediction of the beam lower flange temperature according to EN 1993-1-2 compared to the measured values during the large seventh Cardington fire test, see (Wald et al, 2006)

Guidelines

- EN 1991-1-2: 2002, Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire, CEN Brussels, 2002.
- EN 1992-1-2:2004, Eurocode 2: Design of concrete structures - Part 1-2: General rules – Structural fire design, CEN Brussels, 2004.
- EN 1993-1-2: 2005, Eurocode 3: Design of steel structures - Part 1-2: General rules - Structural fire design, CEN Brussels, 2004.
- EN 1994-1-2: 2005, Eurocode 4: Design of composite steel and concrete structures - Part 1-2: General rules - Structural fire design, CEN Brussels, 2004.
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COST C26 – WG1

Datasheet no. 6

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THERMAL PROPERTIES OF MATERIALS

Description

- In this datasheet the thermal properties of materials are presented according to the appropriate Eurocodes. The materials chosen are: steel, concrete, aluminium and stones.

Technical information and structural aspects

In fire conditions the temperature dependent properties shall be taken into account.

The thermal properties of materials should be determined from the following clauses.

1 Steel

The relative thermal elongation of steel $\Delta l/l$ is given in formulae (3.1 a-c) from EN-1993-1-2). In these formulae the thermal elongation of steel is computed as function of the steel temperature θ_a. EN 1993-1-2 gives formulae (3.2 a-d) for computing the specific heat of steel c_a as function of the steel temperature θ_a. The thermal conductivity of steel λ_a is given by the formulae (3.3 a-b) as function of the steel temperature θ_a. The graphical representation of these formulae is also given for each of the thermal properties.

The thermal conductivity of steel as function of the temperature is presented in *Figure 1*.

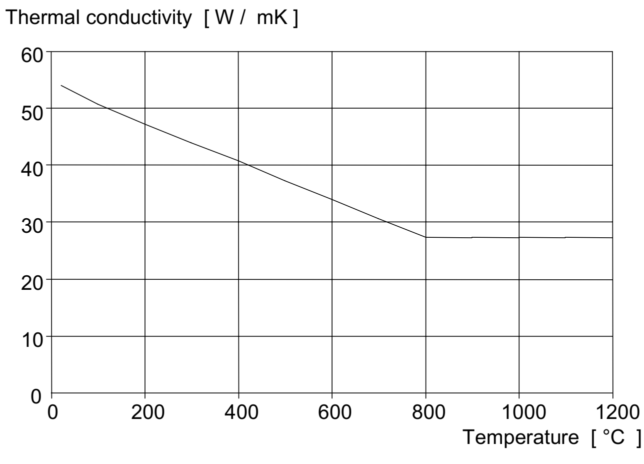


Figure 1. Thermal conductivity of steel at elevated temperature.

2 Concrete with siliceous and calcareous aggregates

The thermal strain of concrete $\varepsilon_c(\theta)$ is given in formulae as function of concrete temperature for siliceous and calcareous aggregates in paragraph "3.3.1 Thermal elongation” (EN 1992-1-2).

The formulae for computing the specific heat $c_p(\theta)$ of dry concrete (u=0%) with siliceous and calcareous aggregates is given in paragraph “3.3.2 Specific heat” (EN 1992-1-2) as function of the concrete temperature. Where the moisture content is not considered explicitly in the calculation method, the function given for the specific heat of concrete with siliceous or calcareous aggregates may be modelled by a constant value, $c_{p.peak}$, situated between 100°C and 115°C with linear decrease between 115°C and 200°C.

The thermal conductivity λ_c of concrete may be determined between lower and upper limit values, given in paragraph “3.3.3 Thermal conductivity” as function of the concrete temperature. The thermal conductivity of concrete is presented in *Figure 2*.

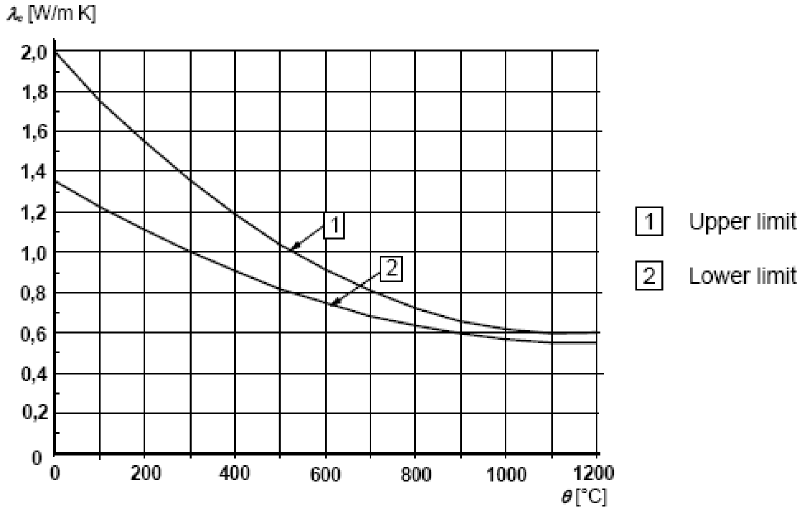


Figure 2. Thermal conductivity of concrete.

3 Aluminium alloys

The formulae for computing the relative thermal elongation (strain) of aluminium alloys $\Delta l/l$ are given in paragraph “3.3.1.1 Thermal elongation” from EN 1999-1-2 as function of the aluminium temperature θ_{al}. The formulae for computing the specific heat of aluminium c_{al} as function of the aluminium temperature are given in paragraph “3.3.1.2 Specific heat”. The variation of the specific heat of the aluminium alloys with the temperature is presented in *Figure 3*. Similarly the computation of the thermal conductivity of aluminium alloys as function of the aluminium temperature is given in paragraph “3.3.1.3 Thermal conductivity”.

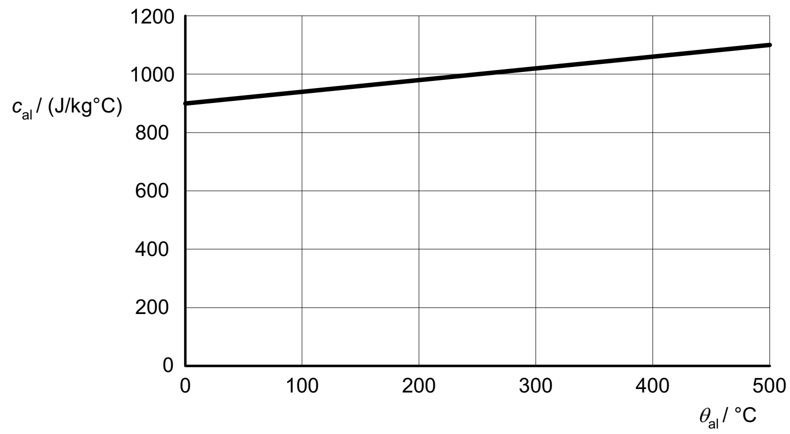


Figure 3. Specific heat of aluminium alloys as a function of the temperature.

4 Natural stones

The heating causes a colour change of stones (*Figure 4a*). Not only colour but also other external signs of heat are observed. Limestone samples are cracked at lower temperatures while at higher temperature the samples collapsed or exploded (*Figure 4b*). According to the thermal decomposition of carbonates this processes is dedicated to the formation of new mineral phases (portlandite).

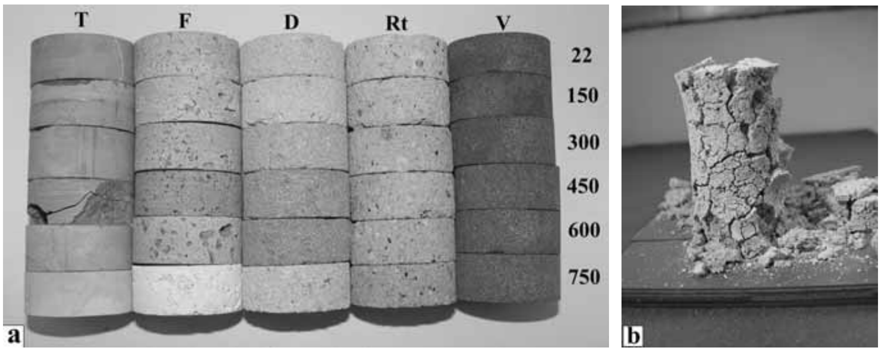
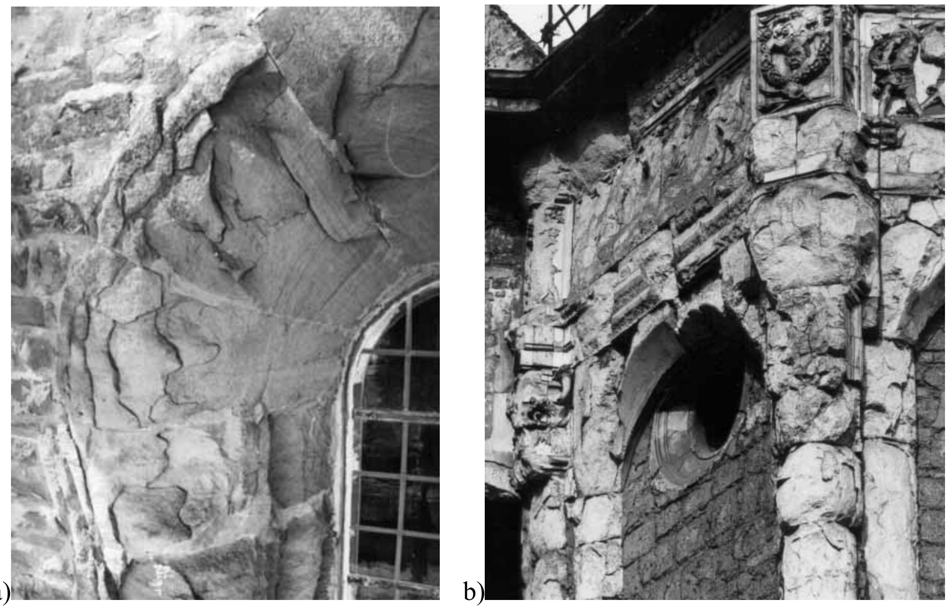


Figure 4. a) Visible colour changes of different stone types before heating and after heating from 150°C to 750°C. T-Tardos compact limestone, F-Süttő travertine, D-Sóskút coarse limestone, Rt-Egertihamér rhyolite tuff, V-Balatonrendesi sandstone, b) Crack formation nad disintegration of cylindrical sample of Sóskút coarse limestone sample after heating on 900°C (after Hajpál 2008)

The most important kind of decay of stones due to fire are scaling off (*Figure 5a*), spalling, cracking, rounding off the edges (*Figure 5b*). Fire can completely destroy ornaments and can damage carved forms. Fire damaged stones are often replaced by new ones (Hajpál 2000).



a) b)

Figure 5. a) Scaling at window edges in Lobenfeld b) Rounding of edges in Dresden

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COST C26 – WG 1

***Datasheet no.** 7*

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GLOBAL MODELLING OF STRUCTURES IN FIRE

Description

Most building structures are required by design codes to be able to resist fire. However, until recently the manner in which fire loading has been handled by codes has been different to the manner in which other loads such as gravity, wind and earthquake have been handled. Whereas it has been normal to ensure structures are able to resist these sorts of loads by means of rationally based calculations, fire resistance is conventionally assessed by reference to a test known as the Standard Fire Test. This test bears little relation to the kind of fires that are likely to occur in real structures and requires that the structural system tested bears very little resemblance to the behaviour of any but the simplest real structures. The shortcomings of the Standard Fire Test have been highlighted by many authors from both fire dynamics and structural engineering perspectives. Despite this, the test (or tabulated results of it) is still widely used for routine structural design. Increasingly, however, designers are recognising the lack of rationality that relying on the Standard Fire Test involves and also finding that the limited range of structures to which it may be applied restricts the opportunities for using economic and innovative structural fire safety designs

Some fire design codes have now introduced the possibility of designing structures to resist fire by calculation. In principle therefore it is now possible for designers to treat fire loading in the same manner as any other form of load. However, for this to happen it must be possible for designers to predict with confidence how a structure will respond to fire. Considerable research effort has been dedicated in recent years to providing the knowledge needed for this and much progress has been made. It turns out that structural behaviour in fire in all but the simplest cases is much more complex than analyses based solely on loss of material strength due to heating can predict. A key aspect of the findings is that treating structural elements, such as beams and columns, in isolation in a fire analysis is insufficient. For accurate results to be produced, either the behaviour of whole structures or the behaviour of parts of structures with appropriate boundary condition must be considered. As a result in all but the most straightforward cases numerical analyses are required to accurately predict the strength and behaviour of structures in fire.

The purpose of this datasheet is to distil the experience gained over the last decade or so of modelling fire affected structures so that new-comers to the field can rapidly appreciate the requirements, challenges and current limitations of global modelling of structures in fire.

Nature of Global Modelling of Structures

Analysing and designing structures for fire loading is a particularly challenging problem for structural engineers. To see that this is the case it is worthwhile contrasting the analysis processes for ambient and high-temperature structural design. At ambient temperature a fortuitous combination of facts regarding loading, material behaviour and design requirements mean that when analysing structures a number of greatly simplifying assumptions may be made. In fire conditions these assumptions no longer hold and both analysis and design become correspondingly harder.

At ambient temperature "actions" on a structure typically result from a combination of wind and gravity loading. Such actions are forces and are (or can reasonably be assumed to be) non-varying when estimating strength. As a result the stresses in structures can be regarded as constant for each load case and it is straightforward to design for sufficient strength. Simplifications may be made as a result of most commonly used structural materials being very stiff. This means that deflections can be considered to remain small and geometric non-linearity can generally be neglected in analyses. In most structures, small deflections are also ensured by serviceability requirements. It is also usually possible to assume either linear elastic or rigid-plastic material behaviour, further simplifying the analysis process by removing the difficulties of handling material non-linearity in calculations. This simplification is even possible with concrete, which is a non-linear material, by use of equivalent stress blocks.

The situation at elevated temperatures is very different for several reasons. The actions on a heated structure are primarily temperatures, or more fundamentally heat-fluxes, that result from exposure of the structure to hot gases and radiation. These produce heating and, subsequent to a fire or as a result of fire-fighting, cooling of the structure. Since not all parts of the structure heat at the same rate, and because structural elements expand when heated, stresses that are not present at ambient temperature are produced within the structure. Whereas the stresses in a structure at ambient temperature may be considered constant, this is not the case in a heated structure because thermal equilibrium will not occur during a typical fire. The inter-play between thermal expansion, restraint to this expansion and the large deflections commonly present in fire conditions, also result in stresses within structural members varying during a heating-cooling cycle. There is no reason why the largest stresses should occur simultaneously with the peak of either the applied heat fluxes or the structural temperatures. A further complication is that heating and cooling will not occur simultaneously in all parts of a structure. This means stresses may be increasing in some areas but decreasing in others.

High temperatures also affect structural materials' mechanical properties with key factors being loss of linearity, strength, modulus and a clear yield point. These changes mean that not only do the stresses within a heated structure change with time but so too does the structure's strength, and this must be considered during analyses. A second consequence of heating is thermal expansion. As noted, if this is restrained in any way, large stresses will result. Thermal expansion also frequently causes large deflections to be present in heated structures. As these deflections are caused by the changing length of heated members it is not necessarily the case, as at ambient temperatures, that they indicate impending failure. Indeed it may be the case that large deflections allow thermally induced stresses to be relieved. However, large deflections do mean it is necessary to account for the effects of geometric non-linearity in analyses if accurate results are to be produced.

The above discussion shows that to get an accurate prediction of structural behaviour at high temperature it is necessary to consider in analyses all the following factors that may typically be excluded or disregarded under ambient conditions: material non-linearity, geometric non-linearity, and time- and temperature-varying strength. If a structure is to be designed to resist fire it is necessary to ensure the structure has sufficient strength and fulfils other design requirements during the entire period it is exposed to temperatures above ambient. The complex and time varying nature of both stresses and strength in heated structures means it is not possible to identify a most serious set of applied temperatures in the same way as a most serious load case can be identified at ambient temperature. Fire loading is a very rare example in structural engineering where all these phenomena need to be considered simultaneously to predict behaviour. Blast and earthquake loading offer two somewhat comparable forms of loading but in these cases other simplifications, such as assuming a lumped mass, may be considered.

The complexity of the behaviour of heated structures has traditionally not been recognized in fire safety design calculations because assessing fire resistance has almost always been done with reference to the Standard Fire Test or has assumed that individual elements of structure may be considered in isolation from each other. In other words, structural fire design has tended to assume statical determinacy. In these conditions high temperature strength calculations only need to account for loss of material strength to obtain a reasonably accurate critical temperature. However almost all real structures contain a degree of redundancy and simplistic calculations will not provide accurate estimates of strength.

Technical information

SOFTWARE

The complexity of even the simplest structural-fire problems means that a numerical analysis will be needed. To date the finite-element method has been used almost exclusively and it seems likely that this will remain the only realistic choice in the foreseeable future. There are, however, choices to be made over the nature of the code to be used. Finite element codes suitable for structural-fire analyses can be broadly divided into two categories: commercial general purpose codes such as Abaqus, Ansys, Oasys, LS-Dyna etc; and research-based codes such as Vulcan, Adaptic and Safir. Commercial codes have the advantages of being able to handle larger problems than research codes and having a wider range of capabilities outside fire engineering. This means that if a structure needs to be analysed for several loading conditions, perhaps fire and seismic loading, only one model would be needed. Commercial programs are, however, costly and normally restrict the ability of the analyst to extend or alter the code. This “black-box” aspect can be frustrating if numerical convergence is not achieved but the reasons for this non-convergence can not be fully investigated. By contrast research codes tend to be much cheaper and the analyst may have access to the code and thus be able to adapt it according to need.

TYPES OF ANALYSIS

There are several decisions to be taken regarding the type of analysis to be undertaken. Depending on the situation any of the following may be required

- An analysis where the mechanical loading is held constant while the thermal loading varies. This represents most fire scenarios reasonably accurately and is the most common form of analysis. Typically the analysis would be broken into two load steps, the first being the mechanical loading and the second the temperature loading.

- An analysis with increasing mechanical loading while the thermal loading is held constant. Such an analysis could be used to find the ultimate mechanical loading for a specific temperature.
- Both mechanical and thermal conditions are time dependent. While probably strictly the case for most structural-fire problems, such analyses have rarely been performed and for most buildings structures this level of detail does not seem to be required. Obtaining accurate estimates of the variation of mechanical loading as a fire developed would be difficult.

There are also various means by which the temperature loading can be represented

- The temperature within the structure can be specified and then a purely mechanical analysis performed. This requires that the temperatures are available either from estimates (perhaps based on simple heat-transfer calculations) or test data.
- The temperature within the structure is calculated based on a finite-element heat-transfer calculation conducted separately to the mechanical analysis. This requires knowledge or an estimate of either the surface temperature of the structure through time or the net heat-flux at the surface of the structure through time.
- A fully-coupled thermal-mechanical analysis. In general this is not required for structural-fire problems as there is normally only weak coupling between heating and stresses/strains. However, there may be special circumstance when it would be needed. One advantage of this approach is that only one analysis is required and so no data transfer from a heat-transfer analysis to a stress analysis need be undertaken.

The most appropriate numerical scheme used for a given analysis must also be selected.

- A quasi-static stress analysis. Here time is a non-physical solution parameter and no time dependent phenomena, such as inertia forces, can be represented. Since inertia effects are not modelled, such an analysis is only appropriate for predicting structural behaviour where structural movement is slow; quasi-static analyses are not suitable for collapse modelling where large inertia forces will be developed. Convergence problems may arise from local buckling instabilities within a large structure with this kind of analysis. Softening and buckling behaviour at ambient temperature is sometimes handled in quasi-static analyses by using an “arc-length” algorithm to solve for load and displacement simultaneously. For analyses with varying temperatures the use of arc-length methods is generally not possible because the applied loads on a structure remain constant.
- A dynamic stress analysis. Here time has physical meaning and so inertia forces can be captured. Dynamic explicit analyses use a conditionally stable numerical scheme which can require small time-steps and so can take a long time to reach a solution. Mass-scaling may be required to obtain a solution in a reasonable time. Full-collapse behaviour can be modelled with this kind of analysis. Since dynamic explicit numerical models will generally reach a solution of some kind (even if it is not a physically meaningful one) without the convergence problems associated with quasi-static analyses, it is recommended that very careful benchmarking is undertaken of such models. This process may include running a quasi-static analysis until inertia forces become significant and comparing the results with the dynamic analysis predictions up to this point.

MATERIALS

Steel

The behaviour of steel at high temperatures is fairly well understood and the usual models used at ambient temperature (von-Mises plasticity with hardening) are still applicable at elevated temperatures. Details of the hardening curves for structural steel are given in a number of publications, notably Eurocode 3. Stresses resulting from thermal expansion are often of great importance in heated structures so the correct coefficient of thermal expansion (which is temperature dependant) should be included in numerical models. This quantity is also included in the Eurocode 3, as are the thermal properties that are needed for heat-transfer analyses.

Concrete

Concrete is a much more complex and variable material than steel, and even at ambient temperatures numerical modelling of concrete structures is less accurate than modelling of steel structures. It is important, therefore, that the limitations of any model used for modelling high-temperature concrete structures are recognized.
There are a range of constitutive models available for representing multi-axial stresses in concrete, most of which are based on some variant of a Drucker-Prager yield criterion. These can be used with uni-axial stress-strain-temperature behaviour from Eurocodes 2 or 4 to provide the means to model behaviour of heated concrete in a basic manner. The Eurocodes also provide information on the quantities such as thermal expansion and thermal properties of concrete. It should be noted that these are dependent on the type of aggregate used.

Many numerical codes allow for a range of phenomena that occur in reinforced concrete to be added to the basic constitutive model, such as cracking, tension softening, load-induced thermal strain, and damaged plasticity. While representing such effects at high temperatures is desirable, it is often not possible to determine suitable input parameters due to lack of experimental evidence. Cracking is the most commonly included phenomenon; it is usual to use a “smeared cracking” approach to representing the cracking behaviour of concrete. This approach assumes cracks are “smeared” over a finite length of concrete and so will not capture the large, discrete cracks that may occur in heated concrete structures. It is an approach that has been shown to be fairly accurate for modelling heated steel-concrete composite structures but its accuracy for other forms of construction that use concrete is currently not clear. Tension softening refers to the behaviour of reinforced concrete in tension after cracking has occurred. It is a complex phenomenon that depends on the interaction between plain concrete and reinforcement. In general it is difficult to capture accurately in numerical models and attempts to do so can lead to numerical instabilities.

Further developments

Research into global modelling of heated structures is continuing. The following is a brief list of areas where research may lead to greater capabilities in the near future, and areas which current modelling techniques are unable to handle.

- Modelling of local effects are often not currently included in models. There is a need to model connections, shear stud behaviour, effect of web openings etc without increasing the size of numerical models to point where they can not be run in a

reasonable timeframe.
- There is considerable work being undertaken at present on improving the representation of concrete behaviour in numerical models. Much of this is focusing on how the important aspects of material behaviour that can not be easily included in current models are best represented. Of particular note are attempts to include load induced thermal strain (LITS) in models.
- The behaviour of structures cooling after a fire, or subject to localized or travelling fires, can be important and is increasingly being considered by researchers and analysts.

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COST C26 – WG1

Datasheet no. 8

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STRUCTURAL MEMBER BEHAVIOUR AND ANALYSIS IN CASE OF FIRE

Description

- The purpose of this datasheet is to summarize the existing knowledge on the topic of the behavior and analysis of structural members in case of fire.
- It summarizes the research contributions from the COST-WG1 members in the study of structural elements in case of fire.

Field of Application

- The analysis of isolated structural elements has been the more used type of analysis in the fire design of buildings, due to the fact that it is much more easy and fast to be made, when compared with a global analysis of the structure.

Research activity

The main research activities in the domain of structural elements, of the COST C26 members have been the study of: class 4 stainless steel box columns in fire (Uppfeldt and Veljkovic, 2007); steel and stainless steel structural elements in case of fire (Lopes et al, 2008, 2007, 2004), (Vila Real et al, 2007a, 2007b); a numerical and analytical model for cellular steel beams (Vassart et al., 2007); Simplified grid model for analysis of reinforced concrete members subjected to fire (Gribniak et al, 2007); and some remarks on the simplified design methods for steel and concrete composite beams (Nigro and Cefarelli, 2007). In this section it is made a brief summary of these research works.

- **1.** Class 4 stainless steel box columns in fire (Uppfeldt and Veljkovic, 2007)

  A study of stainless steel cold-rolled box columns at elevated temperatures is presented, which was a part of a RFCS project "Stainless Steel in Fire". Experimental results of six, class 4, stub columns at elevated temperature, were used to evaluate the FE model. The FE analysis obtained using the commercially available software, ABAQUS, shows that the critical temperature was closely predicted. Further, a parametric study was performed using the same numerical model. This was a basis to check the quality of prediction of a newly proposed improvement for design rules of class 4 cross-sections in fire according to Part 1.4 and Part 1.2 of EC3 (CEN, 2005d and 2005a), stainless steel and fire design part respectively.

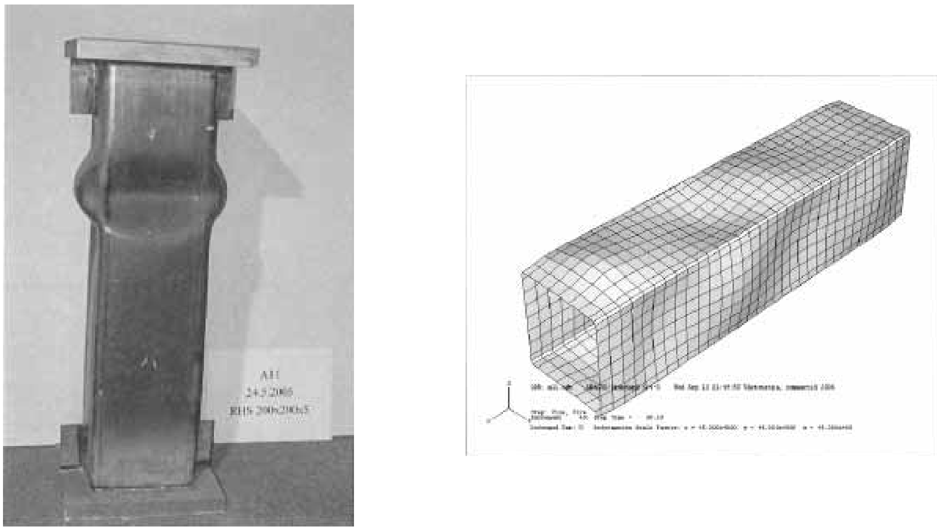

Figure 1. Experimental and finite element tests

The comparison between experiments at the elevated temperature and results obtained from FEA indicated that: assumptions made for the influence of the material properties in the corners are realistic; assumptions for the shape and level of the local buckling, b/200, and global imperfections, L/1000, are consistent with assumptions established at ambient temperature.

The design recommendations for class 4 cross sections made of austenitic stainless steel presented are coherent with part1-2 and part1-4 of EC3. The proposed design model is an improvement compared to the design model on EN 1993-1-2.

- **2.** Steel and stainless steel structural elements in case of fire (Lopes et al, 2008, 2007, 2004), (Vila Real et al, 2007a, 2007b)

Numerical modelling of the lateral–torsional buckling of steel beams at elevated temperature (Vila Real et al, 2007a) has shown that the beam design curve from EN 1993-1-2 is over-conservative in the case of non- uniform bending. An improved proposal was presented that addresses the issue of the influence of the loading type, the steel grade, the pattern of the residual stresses (hot-rolled or welded sections) and the ratio h/b, between the depth h and the width b of the cross-section on the resistance of the beam, achieving better agreement with the numerical behaviour while maintaining safety. A statistical study of the results was performed, showing the accuracy of the improved proposal. (see figure 3a)

Two new formulae for the design of beam-columns at room temperature have been proposed in EN 1993-1-1 as the result of extensive work by two working groups that followed different approaches, namely, a French-Belgian team and an Austrian-German one. Under fire conditions, in EN 1993-1-2, the proposed formulae for the design of beam-columns in case of fire have not changed and are still based on ENV 1993-1-1. In order to study the possibility of having, in parts 1-1 and 1-2 of the EN version Eurocode 3, the same approach for beam-columns, a numerical investigation was carried out (Lopes et al, 2004), with the conclusion that it is possible to use the formulae from the part 1-1 provided that some factors are modified to consider high temperatures (see figure 3b).

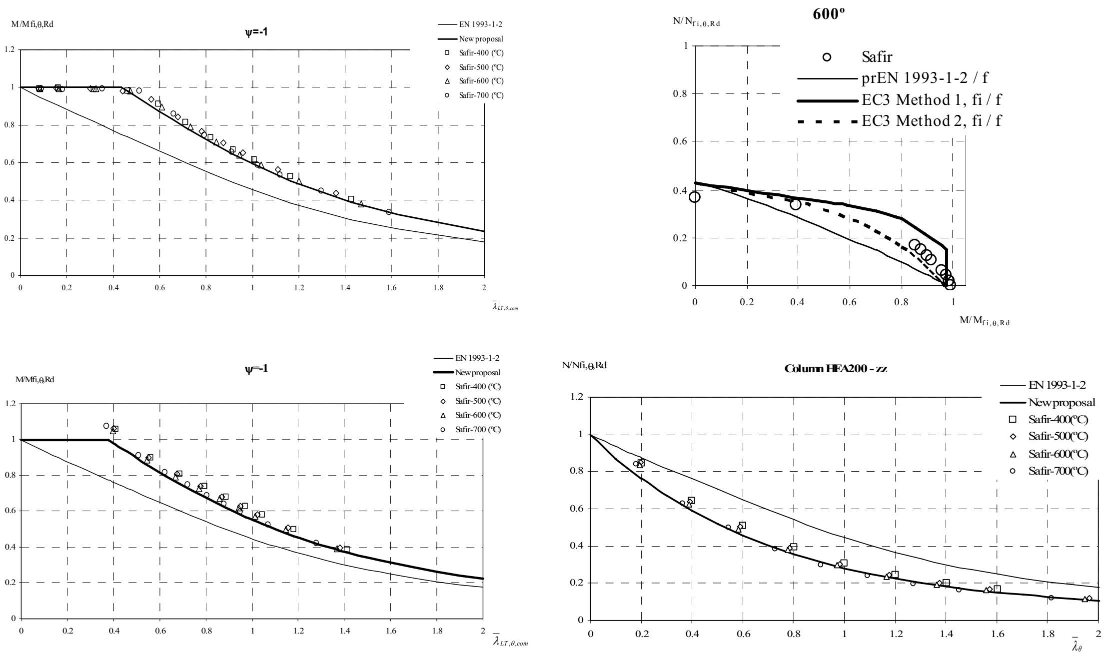

Figure 2. a) Lateral-torsional buckling of steel beams; b) Interaction curves of steel beam columns c)Lateral-torsional buckling of stainless steel beams; d) Buckling of stainless steel columns

New formulae for lateral-torsional buckling (Vila Real et al, 2007b), that approximate better the real behaviour of stainless steel structural elements in case of fire were proposed (see figure 3c), These new formulae were based on numerical simulations using the program SAFIR, which was modified to take into account the material properties of the stainless steel.

It were evaluated the accuracy and safety of the currently prescribed design rules in part 1.2 of Eurocode 3 for the evaluation of the resistance of stainless steel columns (see figure 3d) and beam-columns (Lopes et al, 2008, 2007). This evaluation was carried out by performing numerical simulations on Class1 and Class 2 stainless steel H-columns. It was considered buckling in the two main cross-section axis and, in the case of the beam-columns, different bending moment diagrams. The results presented shown that Eurocode 3 formulae for the evaluation of the fire resistance of columns and beam-columns need to be improved.

- **3.** Numerical and analytical models for the cellular beams (Vassart et al. 2007)

An analytical model representing the web post buckling for cellular beams in case of fire is developed on the basis of that for cold conditions. The analytical model is checked using a finite element model (SAFIR) considering both material and geometrical non-linearity. This model is calibrated on the basis of experimental results (failure modes, stiffness, strength). During fire tests, the main failure mode is web-post buckling. The numerical model is able to simulate the behaviour of composite cellular beams in both cold and elevated temperature conditions with a relatively high accuracy. The analytical model used to evaluate the critical temperature of the web-post gives accurate and safe sided results compared to the experimental tests and FEM model for cellular steel beams. Further improvement must be done in order to take into account the composite cellular beams in the analytical model and define its limits of validity. An example of the numerical model results showing the instability of the web-post and the lateral displacement of the beam and the load-deflection curves are given on Figure 3.

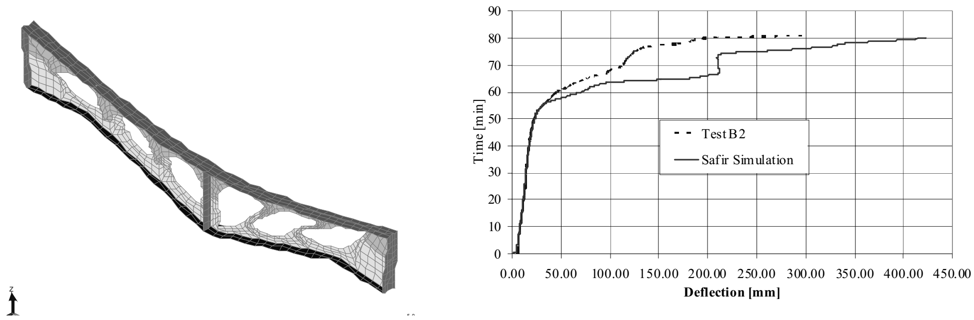

Figure 3. FEM model (cellular beam deformed shape in fire)

- **4.** Simplified grid model for analysis of reinforced concrete members subjected to fire (Gribniak, 2007)

Analytical and computational methods have been extensively developed in the field of RC building exposed to high temperature or accidental fire. Generally an engineer employs various formulae for the fire resistance of structures offered by building codes, without really understanding the thermo-mechanical behaviour of a structure during fire. On the other hand, advanced non-linear mechanical models based on the 2D or 3D finite element (FE) method which were rapidly progressing within last decades are based on universal principles and can include all possible effects. However, such models are computationally too demanding.

This research is aimed at developing a simple computationally effective technique based on formulas of strength of materials and *grid* approach employing temperature dependant material diagrams.

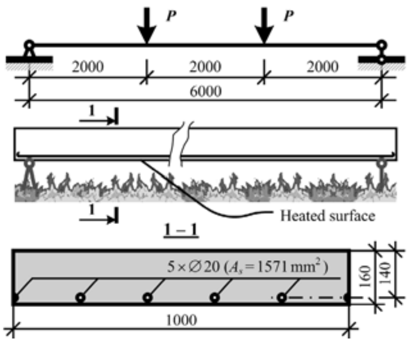

Figure 4. Structural system and cross-section of slab.

The proposed calculation technique is based on the following approaches and assumptions: 1) smeared crack approach; 2) linear distribution of strain within the depth of the section; 3) perfect bond between concrete and reinforcement.

The iterative technique is based on simple formulas of strength of materials. The calculation uses temperature dependent stress-strain constitutive laws of concrete and steel. Thermal strain in a simple and universal way is modelled by the fictitious actions (axial force and bending moment). Thermal creep effect is to be also included in the analysis.

The beam's cross-section is divided into a number of horizontal and vertical layers comprising a grid section. Each grid element may have different material properties.

A step-by-step nonlinear sectional analysis is performed under the external mechanical loads for a given temperature distribution obtained from thermal analysis. Starting with the cross-sectional strains and stresses due to the initial mechanical load a new strain and stress distribution is calculated at any time of the transient thermal analysis.

- **5.** Parametric study of fire resistance of centrically and eccentrically loaded columns (Cvetkovska and Lazarov, 2005, 2004)

  A computational procedure for the nonlinear analysis of a reinforced concrete elements and plane frame structures subjected to fire loading is developed. The program FIRE carries out the nonlinear transient heat flow analysis (modulus FIRE-T) and nonlinear stress-strain response associated with fire (modulus FIRE-S). The solution technique used in FIRE is a finite element method coupled with time step integration.

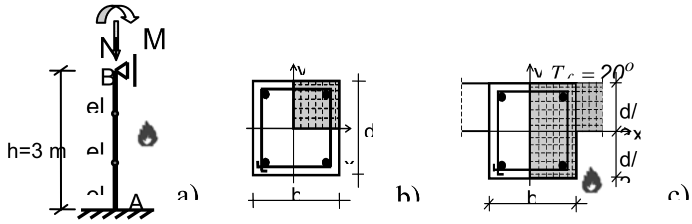

Figure 5. a) Geometry and support conditions. Cross-section discretization when column is: b) exposed to fire from all sides; c) part of a wall for separation the fire compartment

  The influence of: element geometry; support conditions; concrete cover thickness; type of aggregate; compression strength of concrete; steel ratio; intensity of the axial force and different fire scenarios are analyzed and the results are presented. ISO 834 standard fire model is used (recommended in EC2, part 1.2).

- **6.** Some remarks on the simplified design methods for steel and concrete composite beams (Nigro and Cefarelli, 2007)

  This work recalls the main characteristics of a general numerical approach to assess the ultimate bearing capacity of steel and concrete composite beams in fire conditions. It is shown the comparison of the fire resistance between steel beam, composite beam and composite beam with partial concrete encasement.

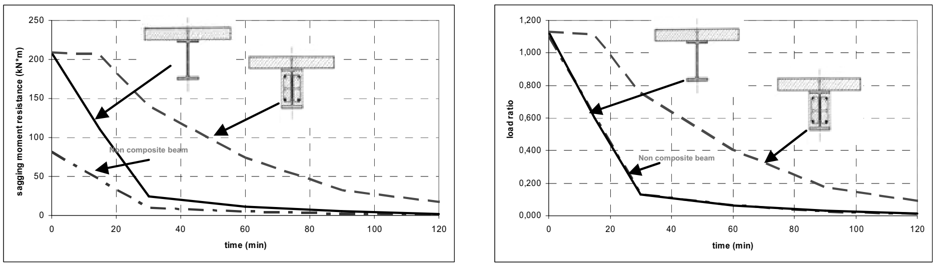

Figure 6. Comparison between various types of beams.

  The following features affecting the resistance of the composite beam with partial concrete encasement are firstly investigated: influence of the beam dimensions and effectiveness of the reinforcing bars in concrete encasement. Moreover, it is shown a comparison between the general numerical approach and the simplified method proposed in EN 1994-1-2 for evaluating the sagging moment resistance of the composite beam with partial concrete encasement. Finally, it is proposed a simplified plastic method for evaluating the sagging moment resistance of the composite beam with partial concrete encasement in fire conditions.

Further developments

The needs for further developments of the WG1 research contributions are:

- Steel and stainless steel structural elements in case of fire

  Developing simple design procedure for columns and beam columns in case of fire, that provides safety and economy, for all stainless steel grades for material properties at high temperatures given in EN 1993-1-2. Study the behaviour of thin-walled (Class4 cross-sections) stainless steel members in case of fire.

- Aluminium structural elements in case of fire

  Validate the simplified calculations methods for the evaluation of instability phenomena on aluminium members in case of fire (lateral-torsional buckling, flexural buckling and beam-columns).

- Numerical and analytical models for the cellular beams

  Simple analytical models for the composite steel-concrete cellular beam based on FEM and tests with the same level of reliability as for steel beams.

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COST C26 – WG1

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STRUCTURAL MEMBER DESIGN IN CASE OF FIRE

Description

- The specificities of the analysis and design procedures, in case of fire, are presented for the commonly used materials in constructional structural elements.
- The materials considered are: concrete, steel and stainless steel, steel-concrete composite, timber and aluminium.
- The approach is mainly based on Eurocode procedures.

Field of Application

- In Eurocodes (European standards developed for the safe, economic and normalized design of structures in Europe) it is allowed to elaborate the fire design of structures on the basis of the analysis of the isolated members.

Technical information and structural aspects

Here it is explained the alternative methods for the fire design of members according to each building material.

According to the fire design part of Eurocode (EC) when a member is considered isolated indirect fire actions are not considered, except those resulting from thermal gradients. Figure 1 illustrates the alternative fire design procedures, given in EC, using member analysis.

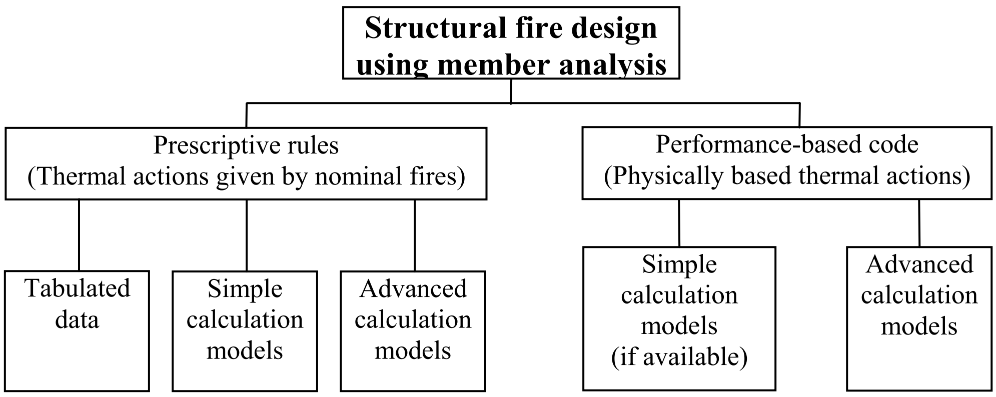


Figure 1. Design procedure

Members are analysed through the determination of the mechanical actions and boundary conditions, and their behaviour can be predicted using tabulated data given for standard fire only if available, in principle data could be developed for other fire curves. It is possible to use simplified calculation methods for standard fire and parametric fire, however the temperature profiles are given for standard fire only, and material models apply only to heating rates similar to standard fire. In EC only principles for advanced calculation models are given. The minimum requirements follow:

$$E_{d,fi} / R_{d,fi} \leq 1.0 \tag{1}$$

where $E_{d,fi}$ is the design effect of actions in the fire situation; and $R_{d,fi}$ is the design load-bearing capacity (resistance) in the fire situation.

In general, the Fire Parts of Eurocodes allow for advanced calculation methods that provide a realistic analysis of structures exposed to fire. Advanced calculation methods may be applied for the determination of the development and distribution of the temperature within structural members (thermal response model) and the evaluation of the structural behaviour. The thermal response model must be based on the theory of heat transfer and take into account the variation of the thermal properties of the material with temperature, and where possible by using effective thermal properties. Advanced calculation methods for the structural response should take into account the changes of mechanical properties with temperature and also, where relevant, with moisture.

- **1.** Concrete elements design according to Eurocode 2 (EC2) part 1.2 (CEN, 2004)

1.1 Tabulated data

EC2 gives recognised design solutions for the standard fire exposure up to 240 minutes. The tables have been developed on an empirical basis confirmed by experience and theoretical evaluation of tests. The data is derived from approximate conservative assumptions for the more common structural elements and is valid for the whole range of thermal conductivity in EC2. More specific tabulated data can be found in the product standards for some particular types of concrete products or developed, on the basis of the calculation method in accordance with EC2.

For load bearing function (Criterion R), the minimum requirements concerning section sizes and axis distance of steel in the tables follows (1).

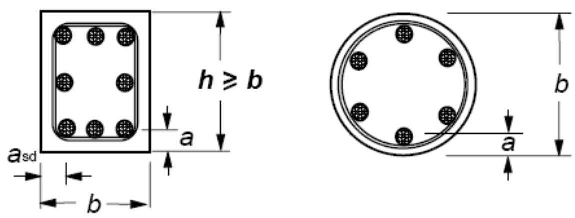


Figure 2. Sections through structural members, showing nominal axis distance a

1.2 Simplified calculation method

Simplified cross-section calculation methods may be used to determine the ultimate load bearing capacity of a heated cross section and to compare the capacity with the relevant combination of actions.

Informative Annex B of EC2 provides two alternative methods, B.1 "500°C isotherm method" and B.2 "Zone method" for calculating the resistance to bending moments and axial forces. Second order effects may be included with both models. The two methods are applicable to structures subjected to a standard fire exposure. Method B.1 may be used in conjunction with both standard and parametric fires. Method B.2 is recommended for use with small sections and slender columns but is only valid for standard fires.

For the "500°C isotherm method" it is considered that concrete subjected to temperatures higher than 500°C, do not contribute for the resistant capacity of the element, while the residual transversal section of concrete keeps its initial values of resistance and of elasticity modulus. This method can be applicable with the ISO fire curve and with the parametric curves.

The "Zone method" is more rigorous when compared to the previous mentioned "500°C isotherm method", in particular for columns. The European standard is applied only to the ISO fire curve. The cross section is divided in a number ($n \geq 3$) of parallel zones of equal thickness (rectangular elements), for each one it is determined the average temperature as well as the corresponding average compressive strength, $f_{cd}(\theta)$, and the modulus of elasticity (if applicable).

Informative Annex C of EC2 provides a zone method for analysing column sections with significant second order effects. Informative Annex D of EC2 provides a simplified calculation method for shear, torsion and anchorage.

Simplified methods for the design of beams and slabs where the loading is predominantly uniformly distributed and where the design at normal temperature is based on linear analysis may be used. Informative Annex E of EC2 provides a simplified calculation method for the design of beams and slabs.

- **2.** Steel and stainless steel elements design according to Eurocode 3 (EC3) part 1.2 (CEN, 2005a)

  The tabulated data normally used are based on experimental tests.

  The verifications using the simplified calculation methods with the ISO curve, can be made in the domain of time, resistance or temperature.

  Simple calculation models are simplified design methods for individual members, which are based on conservative assumptions.

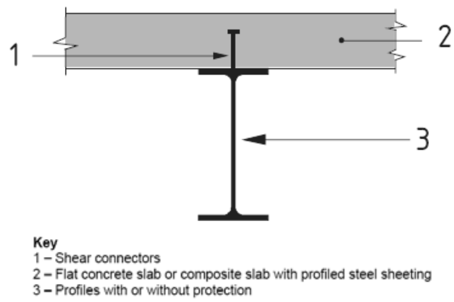
  The design resistance $R_{fi,d,t}$ at time t should be determined, usually with the hypothesis of a uniform temperature in the cross-section, by modifying the design resistance for normal temperature design to part 1.1 of EC3 (CEN, 2005b), to take into account the mechanical properties of steel at elevated temperatures.

  The load bearing capacity of a steel and stainless steel structural members shall be assumed to be maintained after a time t in a given fire if (1) is followed, where $E_{fi,d}$ is the design effect of actions for the fire design situation, determined in accordance with EN 1991-1-2, (the internal forces and moments $M_{fi,Ed}$, $N_{fi,Ed}$, $V_{fi,Ed}$ individually or in combination); and $R_{fi,d,t}$ is the design resistance of the structural member, for the fire design situation, at time t, ($M_{fi,t,Rd}$, $M_{b,fi,t,Rd}$, $N_{fi,t,Rd}$, $N_{b,fi,t,Rd}$, $V_{fi,t,Rd}$ individually or in combination)

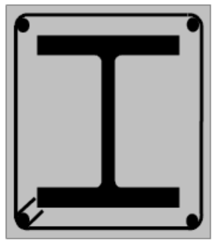
- **3.** Composite elements design according to Eurocode 4 (EC4) part 1.2 (CEN, 2005c)

  3.1 Tabulated data

  The tabulated data referred to member analysis according to EC4 are only valid for the standard fire exposure. The composite members that can be analysed through the tabulated data are: Composite beam comprising steel beam with partial concrete encasement; Composite columns made of totally encased steel sections; Composite columns made of partially encased steel sections; and Composite columns made of concrete filled hollow sections.



a)



b)

Figure 3. a) Composite beam comprising steel beam with no concrete encasement
b) Concrete encased profiles

3.2 Simplified calculation method

Composite beams shall be checked for the resistance of critical cross-sections to bending, vertical shear and connection longitudinal shear.

In the fire situation, test evidence of composite action between the floor slab and the steel beam is available, beams which for normal conditions are assumed to be non-composite may be assumed to be composite in fire conditions.

The temperature distribution over the cross-section may be determined from test, advanced calculation models or for composite beams comprising steel beams with no concrete encasement, from the simple calculation model.

Regarding composite columns, the simple calculation models shall only be used for columns in braced frames. In all cases limits the relative slenderness λ for normal design, to a maximum of 2.

The cross section of a composite column may be divided into various parts.

In simple calculation models, the design value in fire situation, of the resistance of composite columns in axial compression (buckling load) should be obtained from the equation (2) similar to that in normal conditions.

$$N_{fi,Rd} = \chi N_{fi,pl,Rd} \tag{2}$$

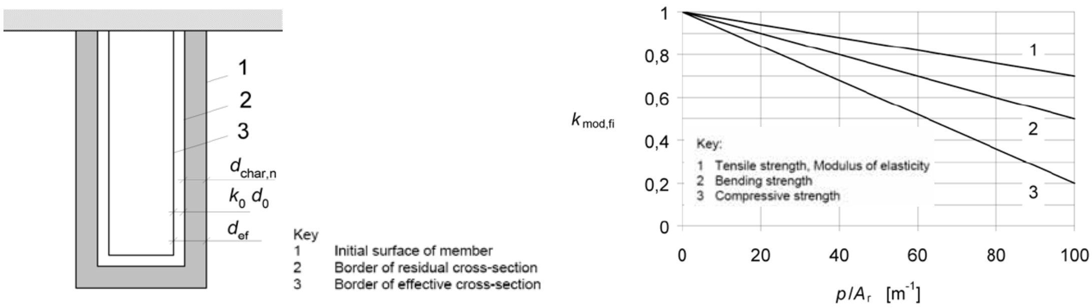
where χ is the reduction coefficient for buckling curve c of part 1.1 of EC3 and depending on the relative slenderness λ (θ) and $N_{fi,pl,Rd}$ is the design value of the plastic resistance to axial compression in the fire situation.

- **4.** Timber elements design according to Eurocode 5 (EC5) part 1.2 (CEN, 2002)

The design procedures for mechanical resistance of timber structures and members are based on those of EN-1995-1-1 for normal conditions. They are combined with simplified rules for determining cross-sectional properties and additional simplified rules for the analysis of structural members and components. The advanced calculation methods may also be used but they shall provide realistic analysis of structures exposed to fire.

The timber exposed to fire burns and develops a layer of char which insulates the solid wood below. The design methods are mainly based on those of normal temperatures using reduced dimensions of the cross-sections and the 20% fractile of the mechanical characteristics of wood instead of the 5% fractile. The charring depth defining the char-line is based on the one dimensional or the notional charring rate. The position of the char-line is the position of the 300-degree isotherm. EN 1995-1-2 gives two alternative methods for the determination of cross sectional properties for the load bearing capacity of beams and columns.

The reduced cross-section method permitting the designer to use 'cold' strength and stiffness properties by reducing the initial cross-section by the effective charring depth. Besides, it takes into account the reduction of strength and stiffness in the heat affected zones by removing a zero strength thick layer (maximum 7 mm) from the residual cross-section. This concept is also applied to small solid timber cross-sections. It is assumed that this zero strength layer is built up linearly with time during the first 20 min of fire exposure, or, in the case of a fire protective layer being applied to the timber member, during the time period until the start of charring. For unprotected members, it takes normally about 20 min to get stabilized temperature profiles in the zone about 40mm below the char layer. Fire tests with protected members have shown that bending stiffness decreases linearly until the start of charring. For simplicity, this linear decrease has been applied to the decrease of the reduced residual cross-section.



a) b)

Figure 4. a) Definition of residual cross-section and effective cross-section and b) Reduction of the mechanical properties.

The reduced properties method gives values of $k_{mod,fi}$ for compressive, tensile and bending strengths as well as the modulus of elasticity of members of softwoods. In EN 1995-1-2 the relationships for $k_{mod,fi}$ are given as functions of the section factor (that is the ratio of the perimeter to the area of the residual cross-section) in analogy with the method used for unprotected steel sections. The reduction of cross-sectional strength and stiffness properties were derived using test results. For small cross-sections with large section factors (and correspondingly high mean temperatures) the curves were fitted to test results on small solid timber frame members in bending.

For the simplified rules for the analysis of structural members and components, EN 1995-1-2 gives a few rules for structural members (beams, columns) and bracing. The purpose of these rules is mainly to reduce the need for verifications. To give an example, compression perpendicular to the grain may be disregarded. The advanced calculation methods for the determination of the mechanical resistance shall be based on fundamental physical behaviour leading to a reliable approximation of the expected behaviour of the structural component under fire. These advanced models may be applied to determine the charring depth, the development and distribution of the temperature within structural member (thermal model) and the evaluation of the structural behaviour of the structure or of any part of it (structural model).

- **5.** Aluminium elements design according to Eurocode 9 (EC9) part 1.2 (CEN, 2006)

The material mechanical characteristics of aluminium reach a zero value (stiffness and strength) at 550°C. The design methods can be applied to elements and structures unprotected, insulated by fire protection material or protected by heat screen. The design methods can be based on simplified models, advanced models or testing. In welded part the weld softening has to be considered. The load bearing function of an aluminium structure or structural member shall be assumed to be maintained after a time t in a given fire if (1) is followed, where $E_{fi,d}$ is the design effect of actions for the fire design situation, determined in accordance with EN 1991-1-2, (the internal forces and moments $M_{fi,Ed}$, $N_{fi,Ed}$, $V_{fi,Ed}$ individually or in combination); and $R_{fi,d,t}$ is the design resistance of the aluminium structural member, for the fire design situation, at time t, ($M_{fi,t,Rd}$, $M_{b,fi,t,Rd}$, $N_{fi,t,Rd}$, $N_{b,fi,t,Rd}$, $V_{fi,t,Rd}$ individually or in combination).

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COST C26 – WG1

Datasheet no. 10

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BEHAVIOUR OF STEEL AND COMPOSITE JOINTS IN FIRE

Introduction

As structural fire engineering is increasingly based on understanding the behaviour of whole structures and their individual parts in fire, this is based on a combination of experimental and analytical research. In the case of joints the motivation for this approach is twofold; on the one hand designers wish to take advantage of the potential control of beam deflections which can be gained from real joints, and on the other it is necessary to ensure that joints are robust enough in fire to resist structural collapse. At the time of writing research into joint behaviour in fire is not sufficiently advanced for detailed design procedures to be developed. This paper gives a picture of the current situation in research and an indication of likely directions of this research in future.

The structural behaviour of steelwork in a building fire is influenced firstly by softening of the material (progressive degradation of the stress-strain curves as its temperature increases), and secondly by thermal expansion. These lead to changes in structural behaviour which interact with other parts of the building; the net structural response can be very different from that at ambient temperature. This depends on the continued integrity of the joints, which experience marked changes in force during a fire, especially in the case of composite floors. As temperatures increase the exposed steel beams initially heat rapidly and expand, with little reduction in strength, and the concrete slab heats more slowly. The resulting temperature difference causes thermal bowing towards the fire, inducing high permanent compressive strains in the steel beams. These increase further as a result of restraint to thermal expansion from the cool structure surrounding the fire compartment. As temperatures increase further, very large flexural deformations can develop, which are acceptable provided that the fire is contained within the compartment of origin. Under these conditions tensile action can develop, particularly within bare steel beams, and the dominant action in the joints is therefore very different from that at ambient temperature, when the moment-rotation characteristics are most important. Once a real fire starts to decay temperatures reduce and the process reverses. The heated beams regain strength and stiffness and also try to contract. However, they have effectively become shorter during the fire, partly as a result of the permanent compressive strains developed during heating, and partly because of the inevitable bending deformation which they have suffered. As the beams try to shorten, they are restrained by the surrounding structure and exert increasingly high tensions on the corresponding joints.

Moment-rotation characteristics at elevated temperatures

Experimental data on the moment-rotation response of joints at elevated temperatures was gathered during the 1990s from full-scale furnace tests (Lawson 1990, Leston-Jones *et al.* 1997, Al-Jabri 1999) conducted on cruciform arrangements. These were reinforced by finite element analyses by Liu (1996), which once again concentrated exclusively on moment-rotation behaviour in the absence of axial thrusts. From the experimental studies semi-empirical rules were postulated by Al-Jabri *et al.* (2004), showing the progressive degradation of strength and rotational stiffness which is illustrated in Fig. 1.

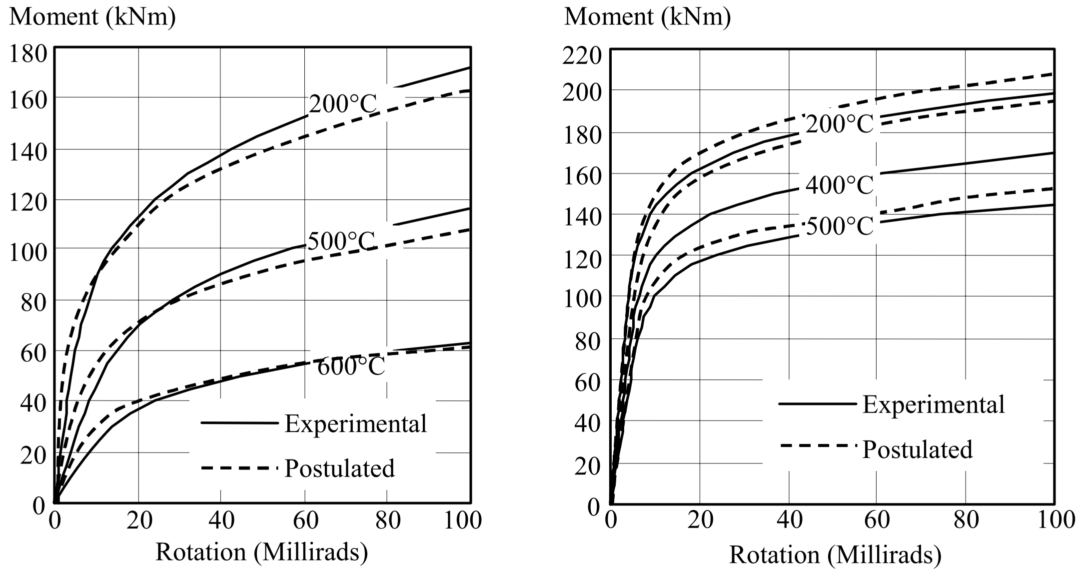


Figure 1. Moment-rotation curves for steel-to-steel and composite beam-column joints (Al-Jabri *et al.*, 2004)

Because of their relative massivity, in terms of a concentration of steel mass with relatively low exposed surface area, compared to the beams and columns to which they are connected, joints tend to heat up in the growth phase of a fire rather more slowly than these members, under equivalent protection schemes. This changes in the decay phase, and joints retain their heat content for longer than these connected members. This means that the rotational stiffnesses of joints at the ends of beams increases as a proportion of the beams' own rotational stiffnesses at these locations, although clearly the absolute magnitudes of both are reducing, as fire temperatures increase. This clearly happens in the Standard Fire conditions under which normal fire resistance ratings are assessed. Thus joints which carry very small moments at ambient temperature begin to carry moments in such conditions which reduce the midspan bending moments considerably; a consequence of this is that midspan deflections are reduced greatly from their simply-supported values. Since standard furnace testing uses limiting deflections as acceptance criteria, this was initially though to be a potential benefit for the fire resistance of beams. In the UK a fire engineering calculation method based on the enhancement of the capacity in fire of composite beams whose joints are designed as simply supported at ambient temperature, because of the hogging moments generated at its joints, was published by the Steel Construction Institute in 1990. In this method the effect of hogging moments at joints in considerably reducing This was widely distributed at the time, but observations of considerable local buckling of the lower flanges of the composite beams next to the connection zones in the Cardington full-scale tests in 1995-96 (Bailey *et al*. 1999) cast some doubt on the safety of this method, and the method is no longer used. It is suggested that hogging moments at joints should not be used to enhance the fire resistance capacity of beams unless very specific detailing is used to prevent the possibility of local buckling of the lower beam flange.

Joint performance in real buildings

The standard joint details shown in the previous datasheet would not all be expected to behave identically in fire. Partial-depth end-plates were used extensively as beam-to-column joints in the Cardington fire tests, and showed considerable evidence of local buckling in the beam lower flanges, in combination with shear buckling of the beam webs, as shown in Figure 2(a).

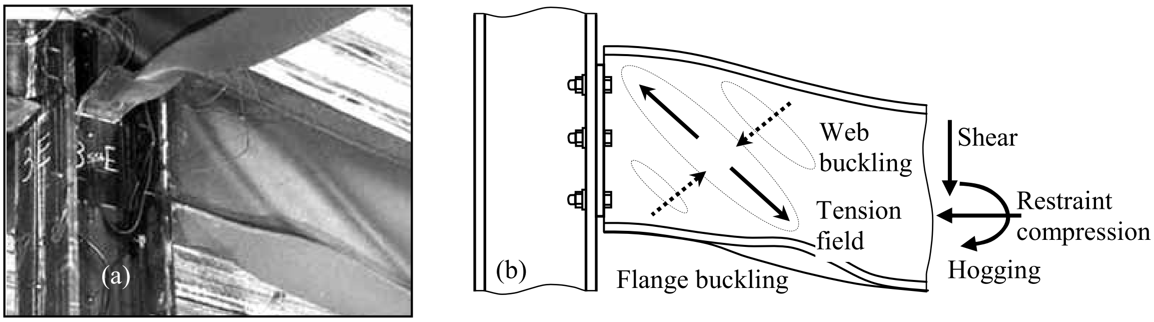


Figure 2. Local buckling; (a) Cardington fire test detail, (b) Forces causing local buckling.

This is seen in the aftermath of most accidental fires in which either steel or composite beams in internal regions of a floor have been subject to the restraint to their thermal expansion provided by continuous areas of slab surrounding these beams. As the steel beam temperature increases this restraint causes high axial compression to grow rapidly in the lower flange and web of the downstand beam, while the expansion is prevented but most of the steel strength remains and deflections are relatively low. The essential limit to this phase comes as the steel strength declines rapidly between 400°C and 700°C, and the axial compression falls. During this phase plastic buckling occurs in the lower flange and web near to the connections, where the axial compressive stresses due to restraint are enhanced (Figure 2(b)) by bending compression stresses due to hogging rotation. When the beam strength reduction becomes very high, it loses most of its bending strength so that deflections increase rapidly, the compressive force reduces rapidly, eventually changing to tension as the steel section begins to carry its loads mainly in catenary action.

This behaviour is illustrated in Figure 3, which shows the axial force changes in the unprotected steel downstand of a composite beam tested at Cardington. It is notable that in heating the tensile force at very high temperatures approximates to the steel strength degradation curve if there is high axial restraint stiffness at the beam ends; however it is reduced by greater axial flexibility at the beam ends or by higher beam deflections. On cooling from this state the beam, which has effectively shortened in restrained heating, contracts against its axial restraint and develops a progressively higher tensile force which again mimics the strength degradation curve. The force generated at this stage is reduced by any axial flexibility, such as low restraint from adjacent structure, low tying stiffness at the end-joints, or the flexibility which is associated with straightening the deflected shape of the beam.

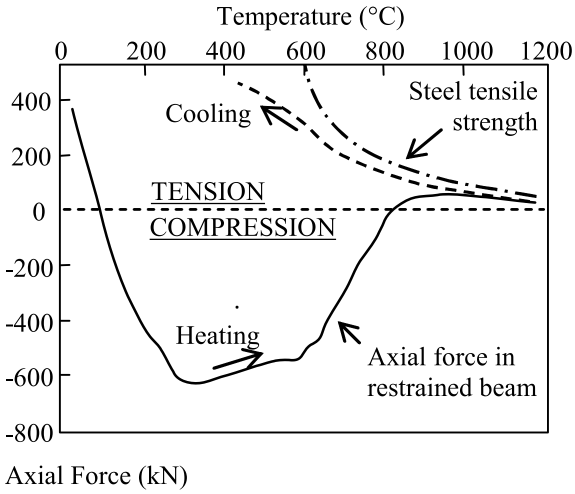
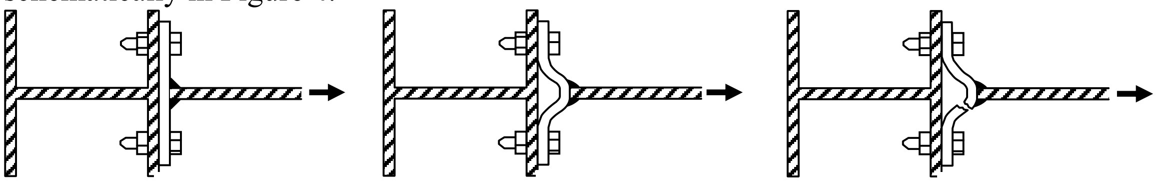


Figure 3. Axial force in steel downstand of axially restrained composite beam.

Observations of structural behaviour in natural fires (Wald *et al*. 2006) and furnace test programmes (Yu *et al.*, 2008; Santiago *et al.*, 2008), have shown steel joints to fail components such as bolts and end-plates because of the high forces induced by the thermal and structural deformations of the connected members. At Cardington, fracture of partial-depth end-plate connections was often observed in cooling. The end-plate material adjacent to the welds on one side of the beam web fractured, as illustrated schematically in Figure 4.



(a) (b) (c)

Figure 4. Partial fracture of end-plates in cooling.

Cases of this partial failure from different fire tests are pictured in Figures 5(a) and 5(c). In no case did both sides of the end-plate fracture, indicating that the increased flexibility produced by the fracture of one side was enough to allow the remaining connection to perform in a ductile fashion. The increased flexibility after fracture allowed the tensile force in the cooling beam to be relaxed through deformation of the joint; the remaining connection performed in a ductile fashion and could still transmit the vertical reaction. An alternative behaviour under these conditions is bolt failure, which very often takes the form of thread stripping within the nuts, as shown in Figure 5(d).

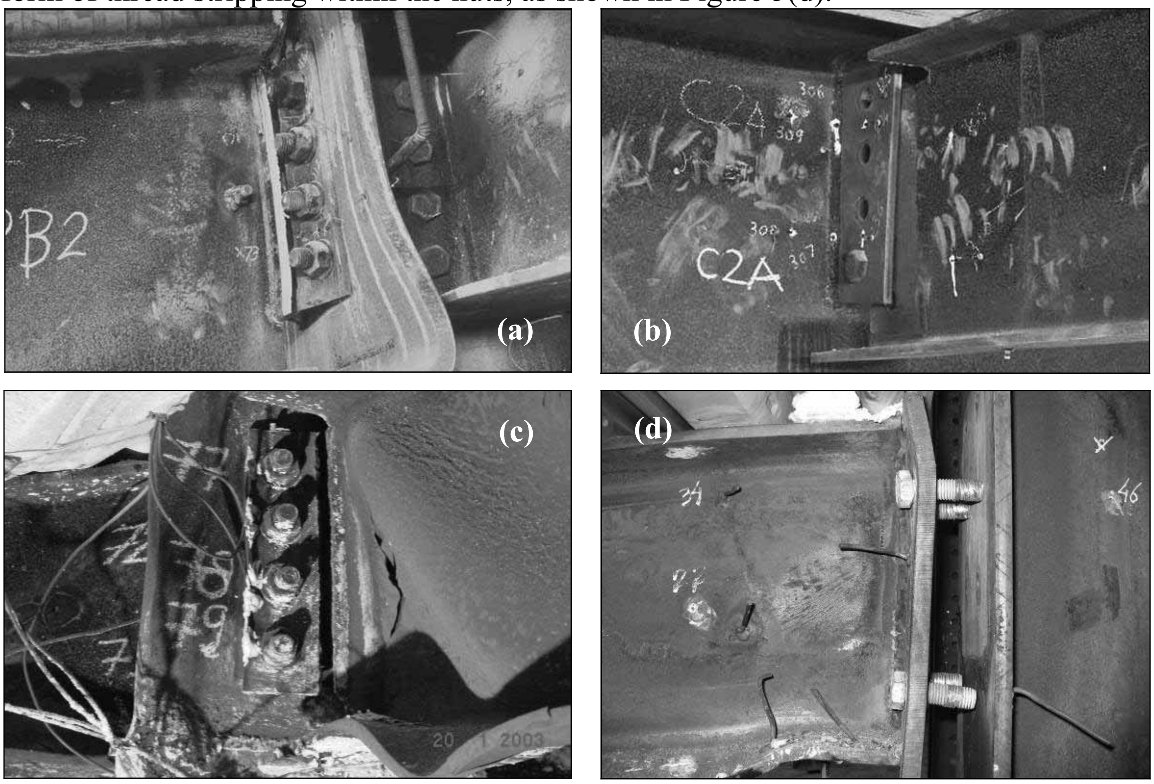


Figure 5. Cardington and Coimbra joint fractures. (a) and (c) Single-sided fracture of partial-depth end-plate in cooling; (b) Shear failure of bolts in fin plate in cooling; (d) Thread stripping of nuts.

Fin plates were used at Cardington to connect secondary beams to their supporting primary beams. In several cases it was observed (Figure 5(b)) that the bolts had fractured in shear at the interface between the fin plate and the beam web. This probably occurred as the secondary beam contracted during cooling, but in other cases might happen as it expands during the heating phase. Fin plates rely on steel in direct tension and shear, and so will always behave in a less ductile fashion than a bending element such as a partial-depth end-plate. However, the rotational stiffness of any end-plate joint is increased considerably when the lower flange of the connected beam makes contact with the face of the column, with a corresponding reduction in rotation capacity.

Measured internal forces in joints

On 16 January 2003, seven years after the end of the main series of Cardington fire tests the seventh full-scale structural fire test was carried out on the same eight-storey steel-framed building. The main purpose of this test was to collect data on the behaviour of typical beam-to-column and beam-to-beam connections subjected to a natural fire. The test was carried out in a compartment on the fourth floor enclosing a floor area 11m by 7m in plan, as shown in Figure 6.

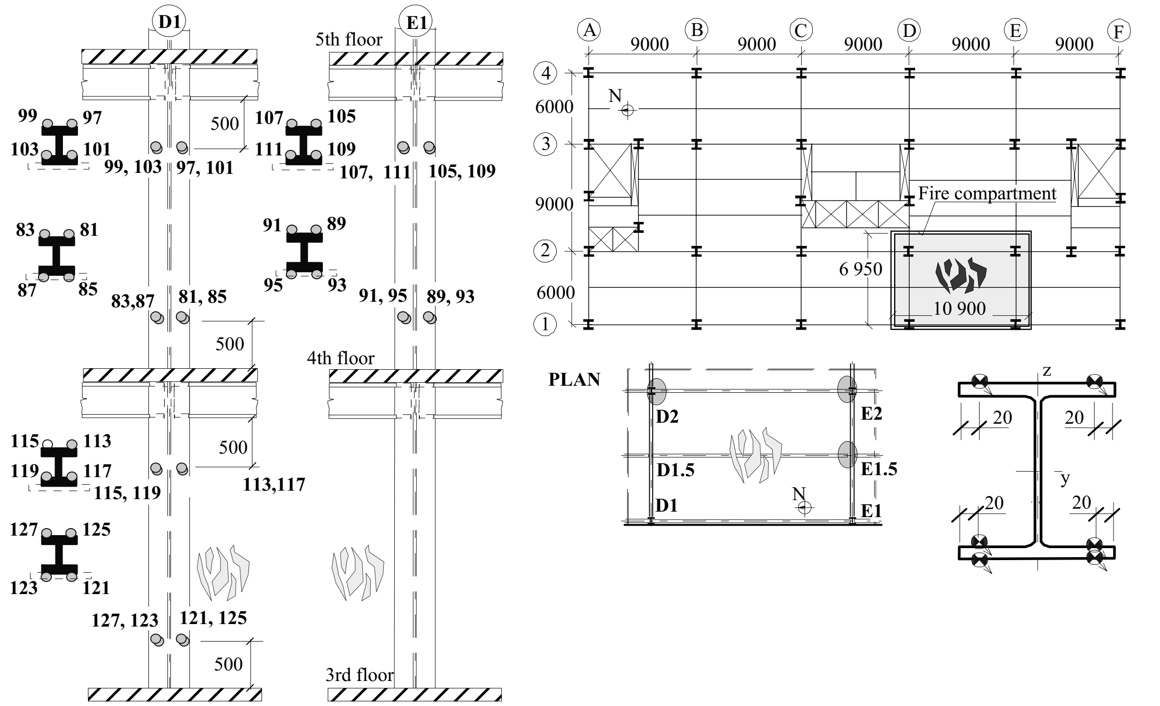


Figure 6. Low-temperature strain gauges for evaluation of forces in connections in the 7th Cardington test.

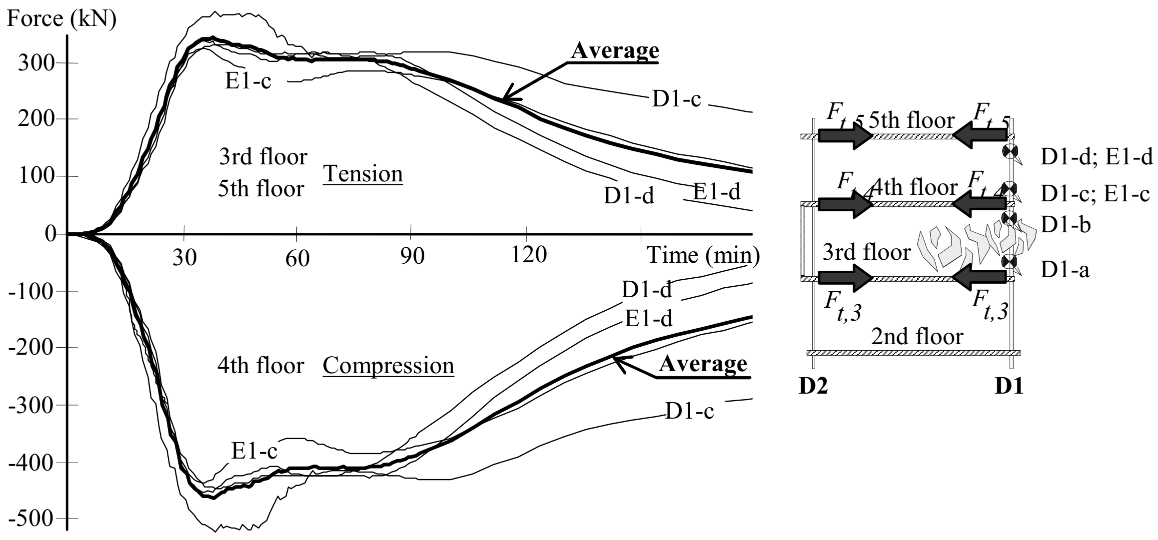


Figure 7. Horizontal forces calculated from the measured bending moments.

Heavy fire protection to the columns prevented excessive increase of temperature, and allowed measurement of strain-gradients, up to 60 minutes on the third floor and for the whole experiment on the fourth floor, using ambient-temperature strain. The stresses in the columns were used to calculate bending moments. The shear forces in the columns were derived from the bending moments, and finally the horizontal forces transmitted through the beam-to-column joints were calculated. On the basis of a continuous beam model, illustrated in Figure 7, the maximum calculated horizontal forces were

$F_{t,3}$ = +344 kN (tension) and -65 kN (compression) at third floor level, and $F_{t,4}$ = -462 kN, compression and +88 kN (tension) at fourt h floor level. Similar results were obtained (Sokol and Wald 2007) during a fire test on a building in Ostrava prior to its demolition.

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COST C26 – WG1

Datasheet no. 11
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COMPONENT-BASED APPROACHES TO STEEL AND COMPOSITE JOINTS IN FIRE

Introduction

When subject to fire steel loses both strength and stiffness. It also expands when heated and contracts on cooling. Thus the effect of restraint to thermal movement of a steel member can introduce high strains in both the member and the joints at its ends. Fire tests on steel structures have shown that the temperatures within joints are lower during heating than those of the connected steel members. This is due to the additional material around a joint (the column, end-plate, concrete slab etc.) which significantly reduces the temperatures within the joint compared to those at the mid-span of the supported beam.

EN 1993-1-2 gives two approaches for the design of steel connections. In the former approach fire protection is applied to the member and its joints. The level of protection is based on that applied to the connected members, taking into account the different levels of utilisation that may exist in the joint compared to the connected members. A more detailed approach is used in the second method, which uses an application of the component approach (see Zoetemeijer 1990) in EN 1993-1-8, together with a method for calculating the behaviour of welds and bolts at elevated temperature. Using this approach the connection moment, shear and axial capacity can be evaluated at high temperatures.

Traditionally most steel beams have been designed for Ultimate Limit State as simply supported. However it has been shown in the large-scale fire tests on the composite building at Cardington in real fires, and in experimental results on isolated connections, that joints which are assumed to be pinned at ambient temperature can provide considerable levels of both strength and stiffness at elevated temperature. This, and the catenary action developed after the loss of most of a beam's bending stiffness, can have a beneficial effect on the survival time of the structure.

Development of component-based methods

The behaviour of the joints as parts of a frame is clearly greatly affected by the high axial forces, created firstly by restraint to the thermal expansion of unprotected beams, and later by resistance to the pull-in of the beam ends when they would normally collapse. The rotational behaviour of a joint is certain to be affected considerably by these axial forces, given the very curvilinear nature of the stress-strain curves for steels, even if local buckling does not take place. Hence, moment-rotation-temperature properties of a joint are not adequate to express the way it will behave in a structural frame in fire. If moment-rotation-thrust surfaces were to be generated at different temperatures this process would require prohibitive numbers of complex and expensive furnace tests for every single joint configuration. The more practical method is to extend the principles of the Component Method of joint analysis and design to the elevated-temperature situation. The basis of this method is to consider any joint as an assembly of individual simple zones, each including several components, as shown in Figure 1. A steel joint under only member end-moment can be divided into tension, compression and shear zones.

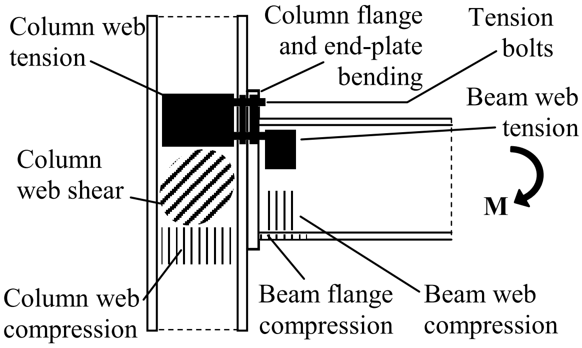


Figure 1. End-plate joint zones under moment.

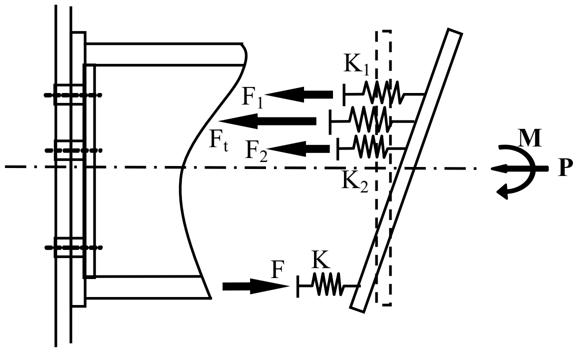


Figure 2. Action of component springs under moment and thrust.

Each of the components is simply a non-linear spring, possessing its own level of strength and stiffness in tension, compression or shear, and these will degrade as its temperature rises. Thus, combinations of moment and thrust are simply different combinations of the horizontal forces in each of these non-linear springs, as shown in Figure 2.

Research studies by Simoes da Silva *et al.* (2001), Spyrou *et al.* (2004a, b), Block (2007), Yu *et al.* (2008), Strejček *et al.* (2008) and Tan *et al.* (2004) have now begun to investigate experimentally and analytically the behaviour of tension and compression zones of steel end-plate joints at elevated temperatures. Simplified analytical models have been developed for the characteristics of some of the main components of flush and extended end-plates at elevated temperatures, and these have been validated against furnace tests and against detailed finite element simulations. Components which have so far been studied in this way are:

- The tension zone comprising the end-plate, top bolts and column flange,
- The compression zone in the column web, under various levels of column axial force,
- The shear-panel zone of the web at the end of the connected beam.

These are sufficient to test the method by using them to regenerate high-temperature moment-rotation characteristics (without axial thrust in the beam or column) which were measured in the earlier furnace tests on cruciform arrangements. The component method has proved very successful in such trials, for example as shown in Figure 3, but for practical application more extensive development is required.

Clearly, each of these component zone studies is to some extent a generic study as well, and it can be anticipated that the models developed will be applicable to other zones with similar characteristics, and to other joint types which employ some of the same components. For example, the model of a beam-end shear panel at should be capable of extension to the column web shear panel. Such simplified models have been shown to be quite reliable for this very common type of joint. The development of similar models for other generic joint types is currently

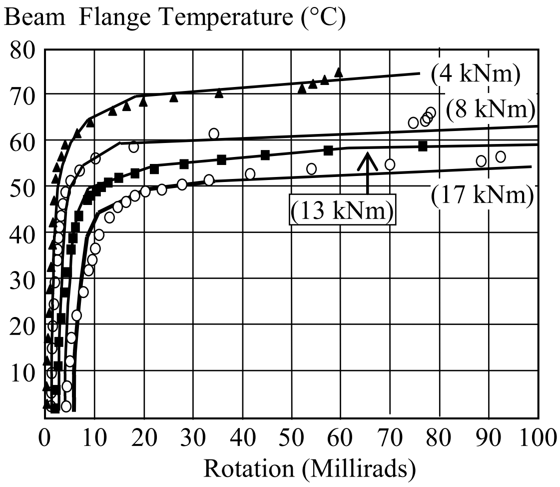


Figure 3. Component-based joint model compared with moment-rotation characteristics from cruciform tests (Al-Jabri, 2004).

less well-established, although models for fin plates and web cleats have been developed (Sarraj *et al.* 2006, Beneš 2006, Yu *et al.* 2008) in recent research.

The influence of the accuracy of temperature prediction on the accuracy of structural modelling is relatively high, as is shown in the comparison of bolt strength reductions given by temperatures calculated on the basis of different assumptions. The measured temperatures of an end-plate connection, measured during the Ostrava fire test, performed in 2006 on a building prior to demolition (Wald *et al.* 2008) are shown in Figure 4.

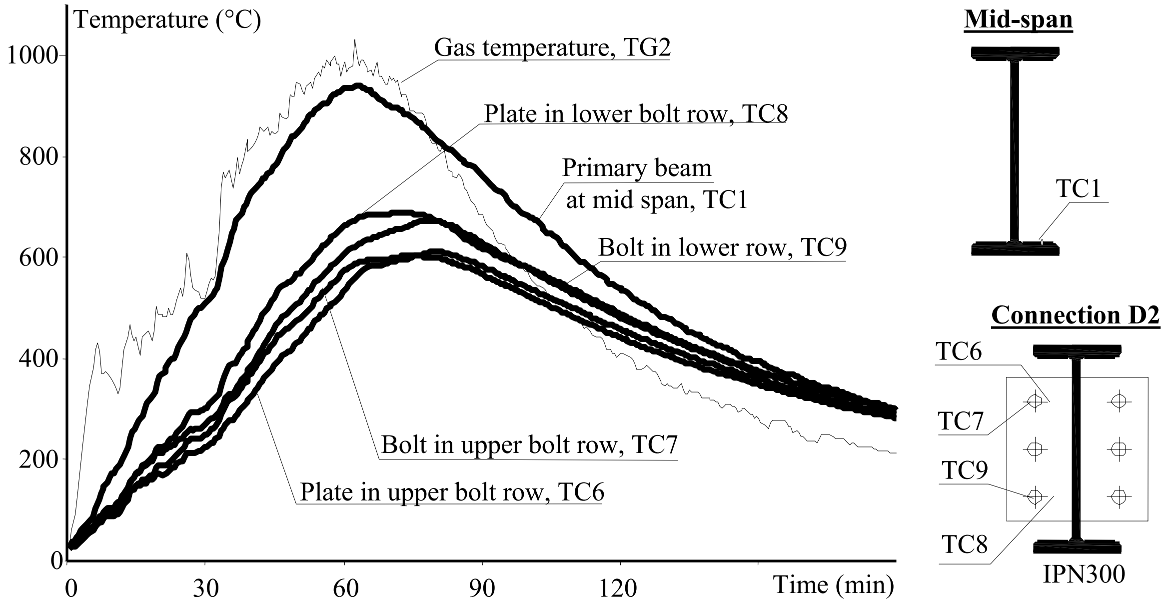


Figure 4. Temperatures measured across height of beam-to-column header plate joint in the Ostrava fire test.

The sensitivity of the prediction may be gauged by the reduction of the bolt resistances based on different thermal assumptions, shown in Figure 5.

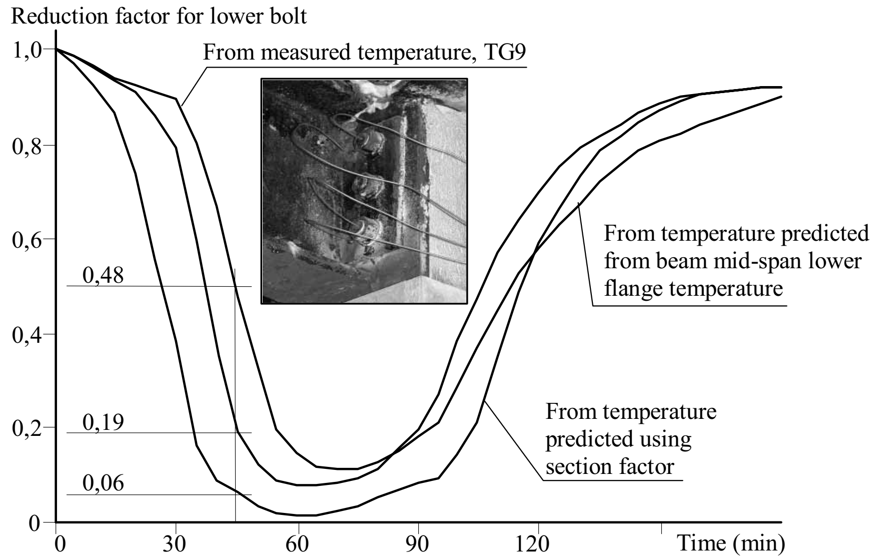


Figure 5. Reduction of the resistance of bolts in the lower row of the header plate connection according to different thermal models, compared to the reduction obtained from the measured temperature.

Significant problems currently need to be solved before component-based connection models can be routinely and reliably used in either simplified analysis or global thermo-structural finite element modelling, to enable connection response to be taken into account in the analytical fire engineering design of steel-framed and composite buildings. One of the more important aspects is the discontinuous nature of the behaviour of many connection components, which is due partly to elastic reverse-straining of components deformed into their inelastic range, partly to the ability of some surfaces to break and regain contact. For example, in Figure 6 top row (spring marked 4) represents the stiffness of the column web in compression only; the two rows below this (springs marked 1, 2, 3) represent the stiffnesses of the bolts and end-plate at these levels, which only have an effect in tension.

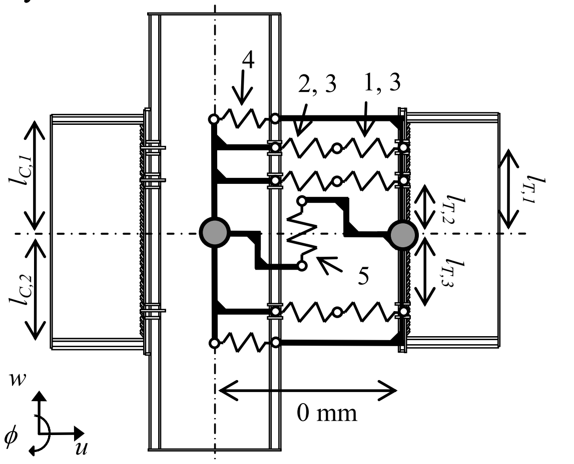


Figure 6. Component-based steel joint model including contact-dependent springs (Block, 2006).

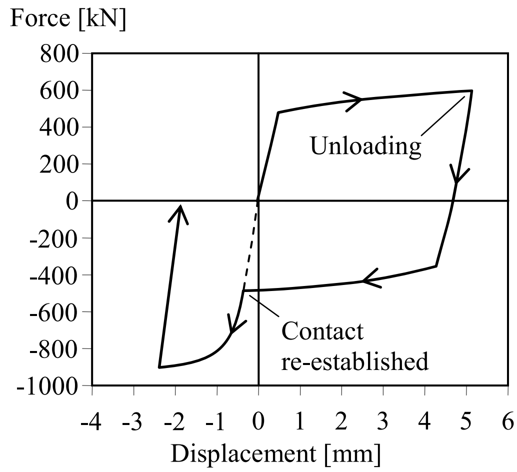


Figure 7. Effects of yield and contact on end-plate in tying (Block, 2006).

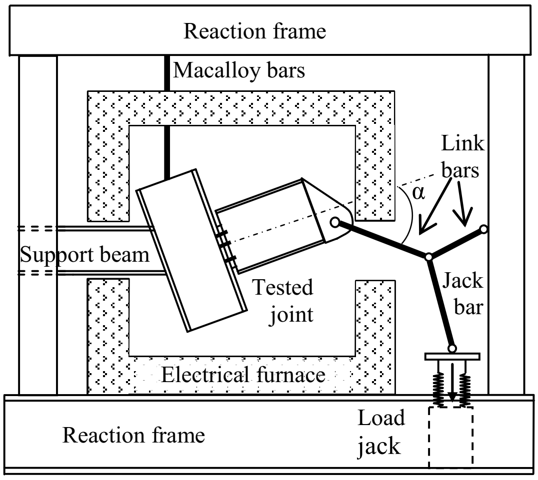
Figure 7 illustrates the complexity of the behaviour which needs to be represented even with very simple connection loading. An end-plate joint is initially loaded to a tying (tension) force of 600kN, then reverse-loaded to a compression of 900kN, and finally unloaded to zero, showing the rapid change in stiffness when surface contact is remade.

Robustness of joints

It is usually implicitly assumed by designers using performance-based fire engineering design approaches that joints retain their structural integrity, yet evidence from the collapse of the World Trade Center buildings, especially Building 5, and full-scale tests at Cardington, indicates that the joints of a building may be particularly vulnerable during both heating and cooling. If joint failure occurs, the assumed structural response may not be able to develop fully, and thereby safety levels may be compromised. It is also important that the fire should be contained within its compartment of origin; the physical integrity of floors needs to be maintained, even at very high distortions. If joints fail, deformations locally are likely to be increased dynamically. Whilst there may be sufficient redundancy within the structural frame to sustain this by redistributing the internal forces, the concrete floor slab has very limited ductility and may not be able to accommodate such deformations without significant cracking, causing loss of compartmentation. Ultimately, joint failure can precipitate local failure of the structural floor system, which may in turn either overload lower floors causing progressive failure, or may allow the supporting columns to buckle, leading to a much more extensive structural collapse. The forces transmitted from one connected member to another across a joint depend on the details of joint behaviour, and without a thorough understanding of this behaviour in fire it is impossible to predict the whole structural performance accurately. Despite this, little research has yet been done on failure of steel joints in fire.

The principal structural effects which would normally be considered as “failure” at joints are fracture due to tension and shear, and local buckling due to compression and shear.

The latter is most likely to occur in parts of the structure which are restrained against thermal expansion. Local buckling of the lower beam flange adjacent to the joint does not in itself constitute a failure in the fire situation, but is known to trigger shear buckling in the web.. The diagonal tension field action caused by this shear buckling has the potential to concentrate the shear and tying forces at the top part of the connection, especially when the joint rotation is very high and catenary tension is developing as a load-carrying mechanism, causing high tying force on the joint. This can trigger a progressive fracture of the connection from the top downwards, which is a genuine structural integrity failure. Depending on the design details, this could involve failure of the bolts, bolt-holes, welds, beam web or end-plate. Even if no fracture occurs during the heating phase, the same progressive fracture could take place during cooling; which may endanger fire service personnel. This joint fracture may lead to a progressive structural collapse, and its avoidance is defined as "designing the building for robustness". Catenary forces reduce as beam deflections increase, and therefore it is desirable for joints to be designed both to retain their integrity and their vertical load capacity while allowing high ductility with respect to rotation and tying deformation.



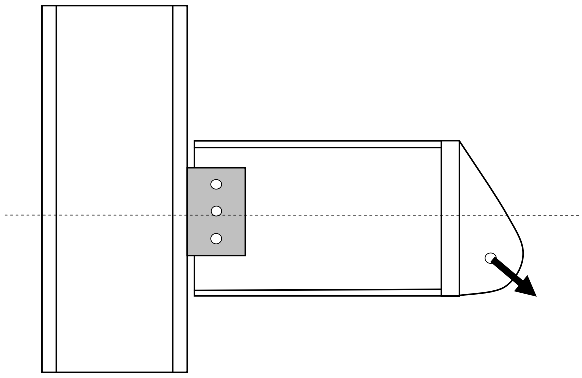
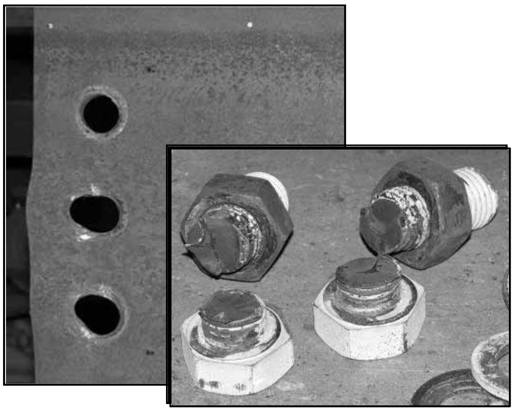


Figure 8. Joint robustness tests; (a) test setup. (b) Fin plate specimen.



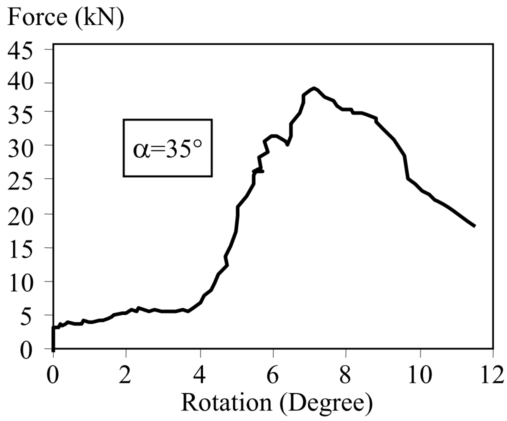


Figure 9. 550°C fin plate test. (a) Deformed beam web holes, (b) Sheared bolts.

Figure 10. Force-rotation curve for fin-plate with 3 Grade 8.8 bolts at 550°C.

The only work done so far on this aspect of joint behaviour in fire has been done recently at the University of Sheffield. Steel joints of four different types (flush and partial end-plates, fin plates and web cleats) have been subjected to combinations of high rotation, shear and tying force at high temperatures using the arrangement shown in Figure 7(a). The results for fin plates (Figure 7(b)) have been published (Yu *et al.* 2008). These typically show low ductility, with sudden "brittle" failure (Figures 9, 10) by sequential bolt shear at fairly low angles of rotation (about 6°), in connections for which ambient-temperature design is based on achieving ductile failure due to bearing. Having seen the effects of real joint failure, and progressive collapse on a catastrophic scale, in both the Twin Towers and other buildings of the World Trade Center, the ultimate strength of joints in severe fires must clearly be high on the research agenda for the next few years.

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COST C26 – WG1

Datasheet no. 12

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DESIGN PROCEDURES FOR STEEL AND COMPOSITE JOINTS IN FIRE

Introduction

The weakest links in the interacting set of components which constitute a framed structure when it is subjected to non-standard loading conditions are usually the joints between the major structural members rather than the members themselves. This is intuitive but is also logical; joints tend to be designed simply to transmit the reaction forces from lower-order to higher-order members, and these reaction forces are assessed on the basis of the normal ambient-temperature Ultimate Limit State loading conditions.

In most previous design documents and research papers the terms "joint" and "connection" have been treated as synonymous, but in Eurocode 3 (EN 1993-1-2, 2005) they have been separately defined. The term "joint" is used to refer to the whole region in which members intersect. It thus includes the plates, bolts and welds, which facilitate the "connection" of the individual elements, and parts of the connected elements in the immediate vicinity, such as the column web and beam ends. Thus "connection" is used to refer to the details of particular connected zones. In broad terms this data-sheet uses this nomenclature, although the distinction is not always clear-cut.

Design considerations for joints

The details of joints used in steel-framed and composite construction vary widely, and depend on factors such as the basic design philosophy and assumptions of the framing system. If a "simple" or "gravity" frame is used then the assumption is that beams are simply supported, with a separate bracing system or structural core resisting horizontal forces on the frame. In this case the basic role of beam-end joints is to carry the vertical end reactions of the beams. If the frame is designed to carry horizontal loads without a separate bracing system then the beam-to-column joints are designed as either rigid or semi-rigid, and must carry combinations of moment and vertical force. In either case the joints adopted will resist both moment and vertical force to some extent, but their design details will be different.

The fire resistance of joints must be at least the same as for the connected members. This means that beam-to-column joints should be able to transmit the internal forces during the whole fire resistance time. When passive fire protection is used on the members this requirement is generally considered to be fulfilled if the same thickness of fire protection is applied to the joints. In consequence of this it is usually said that beam-to-column joints do not present a major problem because, due to the concentration of material, the temperature of the joint tends to be lower than that of the connected members, and therefore their strength increases relative to that of the beams to which they are connected. However this is only superficially logical. The forces transmitted through a joint change massively during the course of heating by fire. In framed structures these usually change considerably because of the effects of restrained thermal expansion interacting with thermal gradients across members and the temperature-related reductions in strength and stiffness of the steel and concrete.

Typical joints at ambient temperature and in fire

Simple joints

For simple braced frames the joints are assumed only to transfer beam-end vertical shear forces into the columns. Such joints should possess very low rotational stiffness so that moments induced in columns are caused only by the eccentricity of the reaction forces from the column centre-lines. Ambient-temperature design procedures for three standard simple joints (Fig. 1) are given in the SCI/BCSA (1992, 1993) "Green Books".

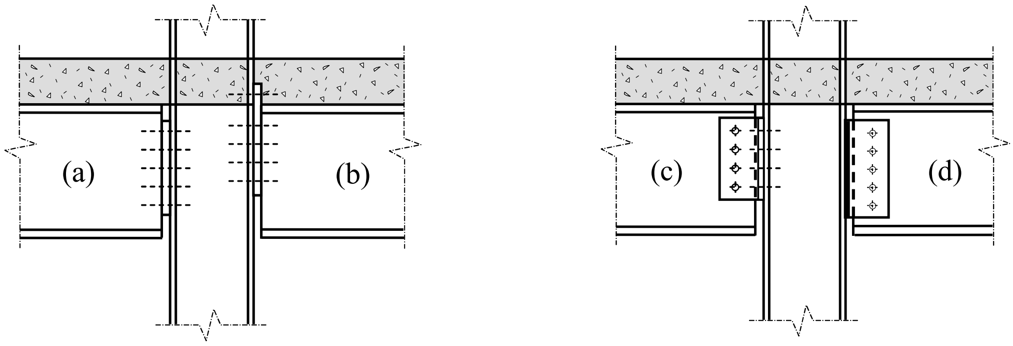


Figure 1. (a), (b) partial-depth end-plates, (c) web cleat, (d) fin-plate.

The flexibility of such joints is purely rotational. Horizontally they may be required to carry a nominal tying force to satisfy national robustness requirements. In fire such joints are required to perform a much more extensive function. The temperature distribution in a joint zone is usually considerably lower than that of the members it connects, especially if some of these members are unprotected, because of the local concentration of material and relative lack of exposed surfaces. As a result of this it is possible to generate moments, even in joints such as these which are designed as simple hinges for ambient temperature, which can very effectively help to resist the deflection of the connected beams. It has previously been thought that this moment-resistance in fire can be utilised in a simple fire engineering calculation for beams, with a residual resisting moment at the beam ends. However, observations of local buckling due to compression stresses in fire have suggested that more caution needs to be applied until further research has been done.

Semi-rigid and rigid joints

Where semi-rigid or rigid joints are needed it is usual to use a variant of the end-plate joint, shown in Fig. 4 (a) and (b). However flush end-plates may be specified by fabricators for convenience, although they have been designed for ambient-temperature strength as simple shear connections, assuming hinged support.

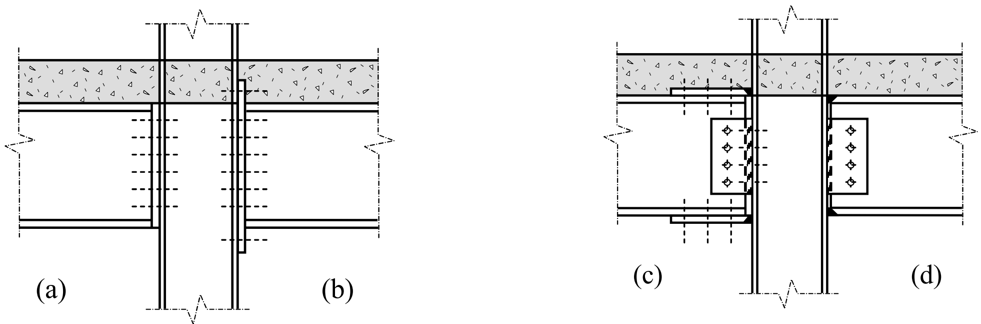


Figure 2. Semi-rigid and rigid joints. (a) Flush end-plate, (b) Extended end-plate, (c) Shop welded/site bolted joint, (d) Site bolted/site welded joint.

Design which assumes full moment transfer at ambient temperature through rigid joints generally implies either extra shop fabrication and expense in stiffening extended end-plates, or site welding. Little research has so far been done on the behaviour of such joints in fire, and it is doubtful whether they could maintain their rigidity, since local buckling is likely to occur at the beam-end adjacent to the connection.

EN 1993-1-2, 2005 3 Part 1.2 Annex D

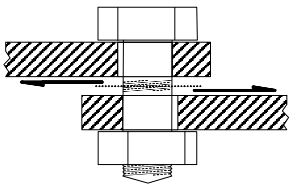
EN 1993-1-2, 2005 has relatively little to say about joints, in contrast to the highly advanced treatment which is possible for joints at ambient temperature under Part 1.8. There is no provision given explicitly for their semi-rigid behaviour, although the relatively cool temperatures in joint components compared with those in the members they connect make the rotational stiffness of simple joints much more significant in fire than at ambient temperature. Annex D is an "informative" section which deals only with simplified connection temperature calculation, and the reduced strength of bolts and welds at elevated temperatures. It does not allow any of the load-deflection behaviour to be predicted.

Bolts in joints

In terms of the member resistance at the connection, there is no requirement to calculate the net section strength in fire, because the fact that the joint temperature is always lower than that of the member away from the joint. This means that the joint becomes stronger than the member in fire, for any of the normal loading conditions for which it is designed at ambient temperature, although both are weakened.

Bolts in shear may either be of the bearing type, in which the connected parts are assumed to be able to slip over one another without significant friction, or of the "friction grip" type which use a specified tension in the bolts to generate frictional resistance.

In the fire case it is assumed that the heating of friction grip bolts has effectively removed the contact pressure, so that they are assumed to have slipped, and they are treated in the same way as bearing bolts.



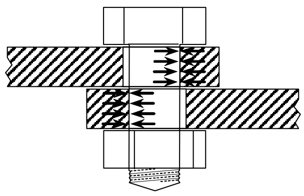


Figure 3. (a) Single shear resistance (b) Bearing resistance

In single shear (Fig. 3(a)) the strength of each bolt is calculated by applying a strength reduction factor, allowing less residual strength at elevated temperatures than the comparable factors for structural steel members, to the design shear resistance of the bolt per shear plane.

In bearing (Fig. 3(b)) the strength of each bolt is given by the lower of the residual strengths at elevated temperature of the bearing resistances of the bolt and the thinner connected plate. The same table of strength reduction factors used for bolt shear applies to the bolt bearing resistance. A reduction of 40% is indicated for bolts in slotted holes.

For bolts in tension, the tensile strength at elevated temperatures is not usually important in calculating the reduced strength of a beam-column or beam-beam joint with respect to the actions for which it is designed at ambient temperature, because bolts are rarely used in direct tension in such joints. However, when beams reach high deflections in fire they lose most of their bending stiffness and strength, and hang in catenary between the joints at their ends. At this stage the tying strength of the joint is probably the key structural property which prevents the floor slabs from collapsing and thus allowing fire to spread vertically to higher storeys. This is a subject in which research is currently active. Both pre-tensioned and non-pre-tensioned bolts are treated in the same way, on the basis that the pre-tension will be lost at high temperatures. The design strength of a single bolt at elevated temperature is again determined from an elevated-temperature reduction of the ambient-temperature strength using the strength reduction factors for bolts.

Welds in joints

For Fillet Welds the reduced strength per unit length of a fillet weld at elevated temperature is determined by applying a tabulated temperature-dependent strength reduction factor to the ambient-temperature strength, which is also a function of the weaker steel grade of the parts joined.

For butt welds the reduced strength of a full-penetration butt weld, for temperatures up to 700°C, is taken as being equal to the strength of the weaker part joined, using the appropriate strength reduction factors for structural steel. For temperatures above 700°C the table of reduction factors for fillet welds can also be applied to butt welds.

Joint temperatures

EN 1993-1-2, 2005 provides simplified temperature calculation methods for joint zones. Three different levels of simplification are allowed.

(i) Temperatures of Individual elements of a connection may be assessed using the incrementally linearised methods provided for the temperatures of protected and unprotected members which are presented in Eurocode 3, using the local A/V values of the elements forming the joint.

(ii) A uniform connection temperature may be calculated as a simplification of the above method using the greatest value of the section factors *A/V* of the connected steel members.

(iii) Linearised temperature distributions may be assessed for beam-column and beam-beam joints in which the beams support any type of concrete floor, in terms of the temperature of the bottom beam flange at mid-span. The beam flange temperature is calculated using the linearised incremental method via a spreadsheet. This is best applied using the section factor for the flange plate itself. When this is determined the temperature distribution through the connection components is represented either as a linear or a bilinear gradient (Fig. 4), depending on whether the depth of the beam is less than or greater than 400mm.

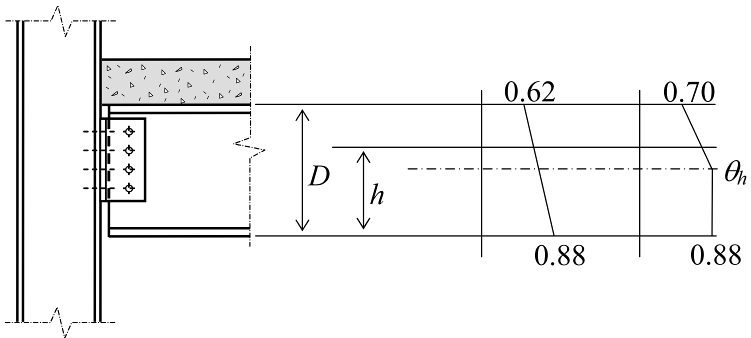


Figure 4. Thermal gradient within the depth of a composite joint.

Eurocode 4 Part 1.2 Clause 5.4

EN 1994-1-2, 2005 cl. 5.4 deals only with shear transfer in connections between composite beams and composite columns, and considers only two joint detail types (Fig. 5(a)). These are bearing blocks and fin plates, both welded to the column face. If bearing blocks are used, they should be detailed so as to guarantee that the beam cannot slip off these supports during the cooling phase of the fire.

It is important to guarantee the required level of shear connection between steel and concrete in fire as well as at ambient temperature. The beam-end shear forces must be distributed into both the steel and concrete parts of a composite column. This can be done using shear studs (Fig. 5(b)), or other appropriate details where the connection is to relatively flexible parts of the steel section of the column, such as flange outstands or the walls of hollow sections. If this is not done these parts may separate from the concrete. Alternatively the steel and concrete parts must be able to fulfil the fire resistance requirements individually. Shear connectors should not be attached to the directly heated parts of the steel sections. For fully or partially encased sections the concrete must be reinforced, with a cover over 20mm but less than 50mm to prevent spalling. Beam-to-column joints should have the same fire resistance time as the member transmitting the actions. One way of achieving this is to apply at least the same fire protection as that of the connected members.

For a beam which is considered as simply supported for normal temperature design, a hogging moment can be developed at the support in fire because of the high beam deflections, if the slab is reinforced adequately to guarantee continuity, and the compression forces generated by restrained expansion can be carried by the steel connection. These moments may be developed for fire resistance periods of 30 to 180 minutes if the gap between the beam end and the column face is less than 10mm, or for beam spans over 5m less than 15mm.

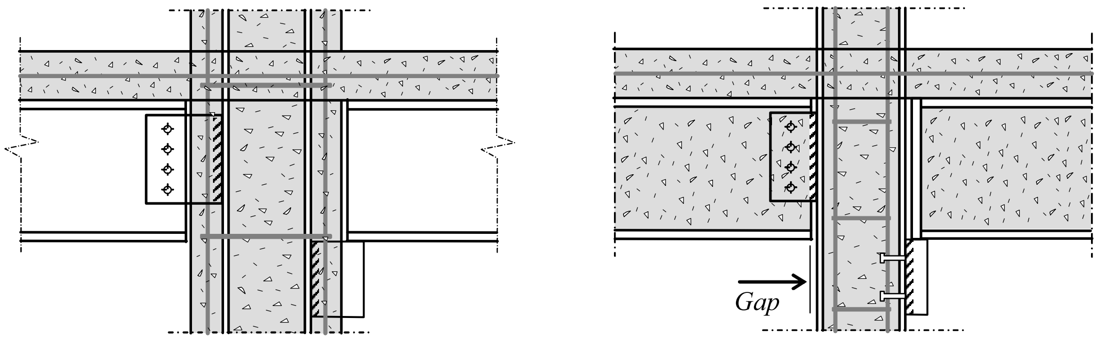


Figure 5. Joints to (a) concrete-encased column, (b) concrete-infilled column.

Connection to concrete-encased column

The practical problem of site connection of steel downstand beams to concrete-encased columns dictates that the only simple connection methods are to use fin-plates or bearing blocks pre-welded to the column face and protruding sufficiently from the concrete encasement to support the beam.

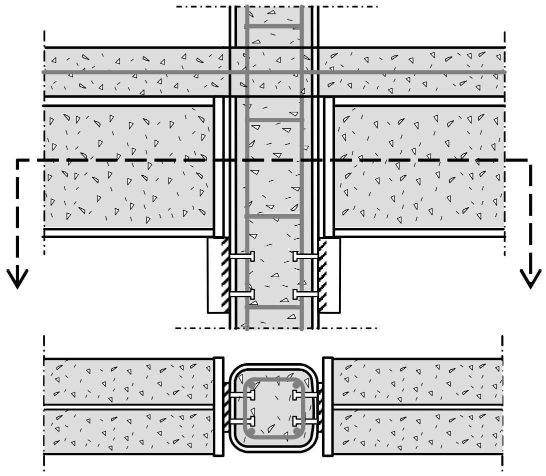
Joints between partially encased beams and columns

Concrete infill between the flanges of I- or H-sections restricts the use of simple connection methods, but it is possible to use both of the above types. Additional shear studs should be provided near to the connection if unprotected bearing blocks are used, because the welds to the column face are exposed to fire. The shear resistance of these studs should be checked assuming a temperature equal to the average temperature of the bearing block. For fire resistance up to 120 minutes additional studs are not needed if the unprotected bearing block is at least 80 mm thick, if it is continuously welded on all four sides to the column flange, or if the upper weld, which is protected against direct radiation, is at least 1,5 times as thick as the surrounding welds and at ambient temperature supports at least 40 % of the design shear force.

Fin plates can transfer shear directly into the web of the steel H-section, allowing it to distribute the stress to the concrete, and so no shear studs are needed. The clear gap between beam and column needs no additional protection if it is smaller than 10 mm.

Joints between composite beams and concrete-filled hollow-section columns

Composite beams may be connected to concrete-filled hollow-section columns using either bearing blocks or fin plates (Fig. 6). The connection details must be capable of transmitting shear and tension from the beam to the reinforced concrete core of the column. With bearing blocks the shear transfer in fire requires additional studs inside the hollow section column, although these must usually be inserted from outside the section through drilled holes before welding. The shear resistance of studs should be checked assuming a temperature equal to the average temperature of the bearing block. If fin-plates are used the best way to guarantee load transfer to concrete and steel is for a single plate to pass through the column, with welds to both walls of the hollow section.



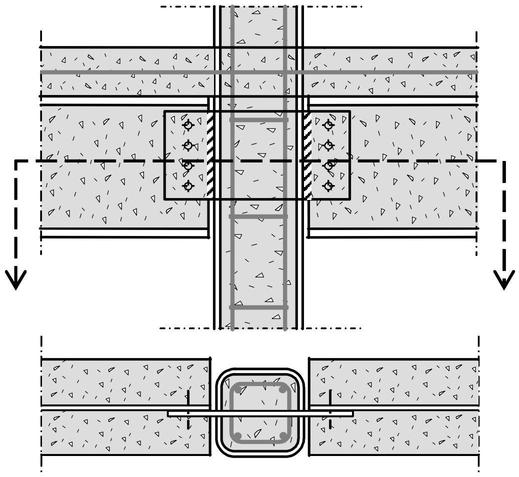


Figure 6. Connections between composite beams and concrete-filled hollow-section column.
(a) Bearing blocks with additional studs.
(b) Penetrating fin plates.

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RISK MANAGEMENT IN CASE OF FIRE AFTER EARTHQUAKE

Description

Fire following earthquake is the most concerning earthquake-related hazard.

Large fires following an earthquake in urban areas are relatively rare phenomena, but they have occasionally been of catastrophic proportions. Building characteristics and density, meteorological conditions and other factors can combine to create a situation in which fire following earthquake is the predominant agent of damage. Records from historical earthquakes show that sometimes the damage caused by the subsequent fire can be much severer than the damage caused by the ground motion itself, this being true for both single buildings and whole regions. For the sake of example, it is estimated that the loss due to the fires after the 1906 San Francisco earthquake is 10 times larger than that due to the ground motion; in addition, within the 1923 Tokyo earthquake, it is estimated that 77% of lost buildings were destroyed by fire. Despite the awareness about this hazard gained on the basis of historical surveys, large fires following earthquakes remain a problem, as clearly demonstrated by the more recent events as the 1994 Northridge (California, USA) and 1995 Kobe (Japan) earthquakes. The seriousness of this problem is mainly due to the probable multiple simultaneous ignitions which fire departments are called to respond to. Such emergency is worsened by earthquake-induced impairing to communications, water supply and transportation, leading to catastrophic scenarios characterized by structural collapses, hazardous materials releases and emergency medical aid.

Field of application

Fire following earthquake is a complex problem, which involves many sequential and situational components. Several subjects are directly or indirectly involved in the related risk management activity, the complexity of the matter requiring a multi-disciplinary approach. It is arguable that, in the first instance, the leading role within the emergency is taken by: fire service; local authorities; other utility organisations; research and hazard informative services. Other interested stakeholders may be: the insurance industry; building owners and/or managers; environmental and community; the general public.

The general approach followed by the hazard and disaster management community during past years has consisted in operating almost solely on the relief programme. So, after the occurrence of a strong earthquake, eventually followed by a large fire, in the perspective of a response-based approach to disaster management, specially trained disaster managers (usually government officials), co-ordinate the relief efforts of both the affected community and the wider aid benefactors. In recent years, however, considering the high catastrophic consequences of a fire following earthquake, relief measures after impact have become increasingly inadequate to protect personal or community assets, safeguard social and economic investments. A new approach is felt, which should be aimed at identifying problems before they happen, by means of systematic process of analysis of risk and decisions about its acceptability. Such general decision-making process is commonly called RISK MANAGEMENT.

RISKS SOURCES are on one hand the damage itself, in particular to pipelines, electric wiring, active and passive fire protection systems, the building structures; besides operating difficulties for firemen: the increase in the time needed to firemen for reaching the place of the fire (due to traffic congestion, collapsed constructions, rubble in the streets, concomitance of multiple fires), the possible difficulties in water supply and the decrease of the collapse time of the structure.

The fire following earthquake risk management requires an approach at two different scales: a local scale, referred to single buildings (BUILDING SCALE), and a global scale, referred to a whole region (REGIONAL SCALE). In both cases, multi-disciplinary approaches are useful.

Technical information

- BUILDING SCALE

  At today's knowledge, the most correct design philosophy for integrating fire safety into the design process for structures appears to be the PERFORMANCE BASED DESIGN.

  Such design approach has already been adopted by International Codes (USA, Australia, UK, New Zealand, Sweden, Eurocode system) in the field of structural fire safety. Performance-based codes for fire safety change the standard of care from meeting the code prescriptive requirements (height and area limits, fire-resistance ratings, egress, separations, etc.) to demonstrating safe performance, which involves design and analysis. As a general rule, the latter could be achieved by a multi-disciplinary approach including fire science, structural engineering and fire safety design tools.

- REGIONAL SCALE

  At the regional level, Geographic Information Systems (GIS) based approaches for earthquake hazard mitigation may be used. Such tools attempt to provide a decision support tool for assignment and routing optimization of emergency vehicles after earthquake, considering the geographic distribution of ignited fires and injuries, locations of emergency response facilities (including emergency operation centres, healthcare facilities, fire stations, police station, etc.), earthquake damage to the facilities and the transportation system.

Structural aspects

BUILDING SCALE

- During the lifetime of a building, a variety of hazards may occur including earthquake, wind, fire, blast and other natural or man-made hazards. Sometimes, several hazards may occur simultaneously or consecutively. In general, they are considered in the design phase of constructions, however there are some scenarios that are still worth to be taken into consideration. For buildings in seismic zone, both fire and earthquake are important design issues, although they are assumed as independent hazards. Actually, the case of earthquake and subsequent fire is necessary to be dealt with, because fire is more likely to be ignited after earthquake causing a very severe damage.
- The exposure of a building to extreme wind, earthquake or gravity loads represents a threat to the building structure, and so the safety of occupants is also strictly related to the safety of the structure itself. On the contrary, in case of development of fire, the

occupants may be directly threatened by smoke and flames, besides by the indirect effect of the behaviour of the building structure under fire conditions. This point involves to consider more stringent design objectives for structures that can be endangered by the combined hazard of earthquake and subsequent fire, aiming at facing the safety in terms of both the behaviour in fire and the direct effects of fires on people within the building. DESIGN OBJECTIVES for fire following earthquake scenarios may therefore include: (1) life safety of the occupants; (2) non-injury of occupants; (3) life safety of fire fighters; (4) non-injury of fire fighters; (5) prevention of damage to contents; (6) avoidance of damage to process; (7) prevention of damage to building; (8) prevention of collapse of building.

REGIONAL SCALE

- The management of the fire following earthquake emergency at regional scale involves understanding and correlating the main aspects of the problem, well keeping in mind that one of the most relevant key factors is time. The essential steps of the process are listed hereafter:

  - *Occurrence of the earthquake*. It may presumably cause damage to buildings and contents, including simple but dangerous knockings of things such as candles or lamps.

  - *Ignition*. Whether a structure has been damaged or not, ignitions may occur due to earthquakes. The sources of ignitions are numerous, ranging from overturned heat sources, to abraded and shorted electrical wiring, to spilled chemicals having exothermic reactions, to friction of things rubbing together.

  - *Discovery*. At some point, the fire resulting from the ignition will be discovered, if it has not self-extinguished yet; however, in the confusion following an earthquake, the discovery may take a very long time.

  - *Report*. If it is not possible for whom discovers the fire to immediately extinguish it, fire department intervention is required.

  - *Response*. The fire department has to respond to the help request.

  - *Suppression*. The fire department then has to suppress the fire. If the fire department is successful, they move on to the next incident. If the fire department is not successful, they continue to attempt to control the fire. The process ends when the fuel is exhausted, namely when the fire comes to a firebreak.

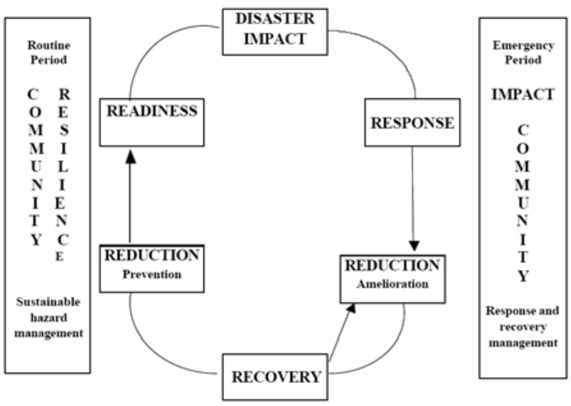

Figure 1. Emergency Management (Source: Britton and Clark, 1999. Adapted for WELG).

- In a long-term perspective, in general the EMERGENCY MANAGEMENT spans *routine periods* as well as *emergency periods*. During the former, the task of the emergency manager is to facilitate sustainable hazard management practices that lead to community resilience. During emergency periods, emergency management focuses on co-ordinating response and recovery requirements. This process is schematically shown in Figure 1.

Research activity and/or guidelines

BUILDING SCALE

Within the structural field, the research about the DESIGN of structures prone to fire following earthquake hazard is oriented at integrating structural fire safety into the design of structural framing systems. This goal can be successfully pursued in a Performance Based Design framework.

As previously pointed out, the structural aspect is one amongst several problems referred to the fire following earthquake risk management, and it is developed separately.

REGIONAL SCALE

- The following most relevant research activities may be mentioned:

  - Much of the early significant studies of fire following earthquake have been led by Scawthorn and colleagues, who has developed models for post-earthquake fire hazard in urban regions that are applicable to both specific earthquakes and for determining annual expected losses on a probabilistic basis (Scawthorn 1986, 1987, 1993). Factors included in these models are building density, wind velocity, deterioration of fire-fighting response and seismic intensity.

  - HAZUS is a hazard assessment modelling programme, developed by the Federal Emergency Management Agency, which assesses damage to the buildings and facilities in a GIS environment. For fire following earthquake, HAZUS follows the logical steps involved in estimating fire losses, in particular the number of ignitions that have the potential to consume one or more buildings, the burned area (which depends on both fire spread rates and suppression efforts) and the population and building exposures affected by the fires.

  - A significant research was done by Cousins et al. (1991), concerning the losses due to post-earthquake fires in central New Zealand. More recently, a model of the spread of post-earthquake fires was elaborated for Wellington City by Cousins et al. (2002). A study done by Botting (1998) suggested various ways in which the fire protection and the fire engineering measures may reduce post-earthquake fire losses in urban regions. The research was based on the analysis of fifteen major earthquakes and recommendations were given on the fire brigade response, urban water supplies and urban macro-scale fire protection.

  - The life-safety risk for post-earthquake fires from the perspective of gas and electricity distribution systems was investigated for the Pacific Earthquake Engineering Research Center by Williamson and Groner (2000). The study, based on fire scenarios relating to the various aspects of gas and electricity distribution systems, concluded that the old residential buildings susceptible to structural damage and potential collapse pose an increased risk due to the concurrent fire hazard. In relation with the lifelines services, a study by Robertson and Mehaffey (2000) recommends that performance based building codes should contain a framework to prevent undue reliance on sprinkler and other life safety systems that are dependent on seismically vulnerable water and electrical services.

  - A book published by ASCE (Scawthorn et al, 2005) gives the most comprehensive view on fire following earthquake in urban regions, covering the history of past post-earthquake fires, models for fire spread in post-earthquake urban environment and cost effectiveness of various fire after earthquake mitigation strategies.

- Australia and New Zealand became, in 1995, the first Countries in the world to formally develop and adopt a GENERAL STANDARD on Risk Management. The Australian and New Zealand Risk Management Standard (AS/NZS 4360:1999 2nd Edition) provides a formalised, systematic decision-making process by which identifying solutions to issues concerning the vulnerability to natural hazards.

  Three features of the Standard may be highlighted. They are: (1) the definition of risk management; (2) the process of risk management; (3) the context within which the risk activity takes place.

Example of application

BUILDING SCALE: PBD APPLIED TO SINGLE BUILDINGS

Chen et al., 2004

- Proposal of a procedure consisting in four major steps: (1) hazard analysis, (2) structural and/or non-structural analyses, (3) damage analysis and (4) loss analysis.
- One of the key points in this procedure is that structural and/or non-structural analyses are repeated because the earthquake and the following fire are two different hazards occurring sequentially. After the earthquake, the actual status of the building needs to be evaluated. In particular three kinds of damage should be considered: damage to structures, damage to fire protection of structural members and damage to non-structural fire protection system. Re-evaluating the fire hazard is also very important because the damage to the fire protection systems may affect the development of the fire hazard.

Johann et al., 2006

Framework developed to integrate structural fire safety into the design of structural framing systems. It consists in a series of flowcharts to identify and organize the specific functions for involvement of fire performance expertise, including design by calculation. In the perspective of PBD fire safety integration into the structural design process, five activities are considered. They are the following ones: (1) Structural design for gravity and lateral loads; (2) Modifications during service life; (3) Definition of design fire conditions within the building; (4) Analysis of structural response to the design fire conditions; (5) Evaluation of the acceptability of the predicted performance.

REGIONAL SCALE: RISK MANAGEMENT *AS/NZS 4360: 1999*

The outline of the risk management process is organized as follows: Establishment of the strategic, organizational and risk management context; Risk identification; Risk analysis; Risk evaluation; Risk treatment; Monitor and review; Communication and consulting.

Further developments

Further studies are needed, mainly focusing at achieving the following primary objectives:

1) With regard to the PBD approaches for single buildings, the suitable definition of performance criteria and design procedures has to be consolidated;

2) With regard to the GIS based regional approaches, the predictions of the PGA-fire occurrence correlations should be refined.

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STRUCTURAL ANALYSIS AND DESIGN IN CASE OF FIRE AFTER EARTHQUAKE

Description

The fire following earthquake is a low probability high consequence hazard. According to the Directive of the European Commission issued on 21 December 1988, 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 assured for a specific period of time, the generation and spread of fire and smoke within the building and to neighbouring constructions are limited, the occupants can leave the injured place by themselves or be rescued by other means and the safety of rescue teams is considered. Consequently, the behaviour in fire of structures which have been damaged by earthquakes represents an important investigation field since in many cases fires break out after a seismic event, giving rise to a real catastrophe. In fact negative effects of fires on structures and human lives may be comparable to and even more important than those of the earthquake itself. Moreover, even in case no fire develops immediately after an earthquake, the possibility of delayed fires affecting the structure must be adequately taken into account, since the earthquake-induced damage makes the structure more vulnerable to fire effects than the undamaged one. This is because the consequence of fire on a structural system is mainly a gradual decay of the mechanical properties as far as temperature grows. It is apparent that the more the structural behaviour is degraded after an earthquake the more time up to collapse due to fire is short.

Field of application

- Large urban areas prone to severe earthquakes may be susceptible to undergo catastrophic conflagrations following the shaking event.
  The likelihood of fire following earthquake leading to a catastrophic scenario is related to the presence of RISK FACTORS (RF), which can be grouped with reference to the possible phases of the phenomenon:
  - RF related to the direct *earthquake effects* are: damage itself, displacement of dangerous contents, fracturing of gas and / or electricity connections and / or reticulations;
  - RF related to the *sources of ignition* are: open fires, hot surfaces, boilers, short circuits from structural damage, fallen live wires;
  - RF related to the *establishment of fire* are: fuel, failure of active suppression systems within buildings (like sprinklers);
  - RF related to the *spread of fire* are: high density of buildings, boundary barriers not designed to modern fire spread resistance (for instance, windows), direction and velocity of wind, damage to passive measures;
  - RF related to the *detection / containment / extinguishment* of fire are: uncertainty of fires location, impairment of fire brigade response, loss of water pressure due to reticulation damage.
- A comprehensive methodology of performance-based design of buildings in the fire-safety codes, which should distinguish between structures located in seismic and non-

seismic areas, by requiring more stringent fire resistance provisions for those buildings potentially subjected to seismic actions, should be developed.

Technical information

- The common trend in structural engineering which integrates the PERFORMANCE BASED DESIGN approach gives the opportunity to incorporate fire safety engineering into the design process for structural framing systems. This requires an interdisciplinarity that considers fire science, structural systems and materials and fire safety design tools.
  International Codes (USA, Australia, UK, New Zealand, Sweden and Eurocode system) have already adopted performance-based approaches to structural fire safety.
- One of the main concerns and source of aleatority for the evaluation of the structural behaviour in fire and the protection strategies, aiming at the design, is the MODELLING of the FIRE event, which can be carried out by following several methods, affected by different levels of refinement and complexity. The most spread models are the following ones:
  - The *nominal standard temperature-time ISO 834 model*. It does not take into account any physical parameter, and can be far away from reality. From the beginning, the nominal model supposes that the entire compartment is in the flashover phase and the temperature is increased continuously, without taking into account the cooling phase.
  - The *parametric fire model*. It considers the cooling phase and gives the temperature-time curve function of the fire load density and openings. This model is, however, limited to the surface and the height of the fire compartment considered, and supposes that the temperature is the same on the entire compartment, from the beginning of the fire.
  - The *combined "Two Zone" and "One Zone" model*. It is a modern fire model approach. In this natural fire model, during the pre-flashover phase, the fire compartment is divided in a hot upper zone and a cold inferior one. For each zone, with uniform temperature, mass and energy equations are solved. After the flashover, the temperatures is considered uniform and it is determined by solving the equations of mass and energy of the compartment, taking into account walls and openings.
- Different ANALYTICAL TOOLS are available for the assessment of the structural response in fire. For the sake of example, it is possible to use numerical programs able to perform only the analysis in fire, resulting in the evaluation of the fields of temperature within the structural members. This requires the use of other programs for the structural analyses, for the evaluation of both the stress and strain states taking into account the temperature variation. Currently, some programs, able to carry out the temperature and displacement analyses in a unique structural model, are available.

Structural aspects

- Structural systems may serve two main functions during a fire event:
  1. To continue to support loads:
     building occupants can exit safely and fire-fighters have sufficient time to respond and control the fire.
  2. To serve as a barrier and/or support other barriers to fire propagation:
     - the collapse of a floor could allow a fire on one storey to spread into another storey.
     - excessive deflection of a floor may contribute to the instability and failure of a partition wall, which could allow the fire to spread into adjacent compartments.

- Accordingly, FIRE PERFORMANCE CRITERIA for a structural system may include:
  - Limitations on member deformation or requirements for serviceability;
  - Requirements for load-carrying capacity - prevention of collapse;
  - Time to failure requirements - to allow occupant egress and suppression activities;
  - Fire containment requirements - limitations on the impact of a fire on structural members distant from the fire and prevention of room-to-room fire spread.
- The ASSESSMENT OF A STRUCTURE'S RESPONSE TO FIRE requires the ability to analyze the effects of fires on:
  - individual structural members and connections,
  - assemblies of members,
  - entire structural frames,
  - interactions between components.

  It is evident that the capability of catching the structure's response to fire depends on the used analytical tools.
- Several CATEGORIES OF STRUCTURAL MODIFICATIONS occurring during the service life of a building should be considered in the design process, in order to consider the structure "as it is", during its lifetime. As an example, the following ones are worth to be noticed:
  - Presence of passive protection of structural members by means of insulation, coatings, barriers, etc.;
  - Differences between the originally specified structural configuration and/or protection and the as-built condition;
  - Normal operation and deterioration: rust, corrosion and other environment-related deterioration mechanisms, as well as aspects of building operation that may cause inadvertent damage or long-term wear to structural members or their fire-protective insulation or coatings;
  - Changes to the structural configuration and/or protection caused by pre-fire events such as earthquakes, blasts, accidental loss of protective material, etc.;
  - Changes to the structural configuration and/or protection subsequent to a fire.
- STRUCTURAL ANALYSES IN FIRE deeply depend on the modelling assumptions. In this perspective, a very important role is played by the modelling of *earthquake-induced damage*, of *material behaviour at elevated temperature*, as well as the modelling of *fire*.

Research activity and/or guidelines

Main issues as developing subjects of the relevant research activity are summarized hereafter.

- ASSESSMENT OF POST-EARTHQUAKE FIRE BEHAVIOUR OF STRUCTURES
  Evaluation of the effects of earthquake-induced damage on fire resistance and collapse modes: the more the structural behaviour is degraded after an earthquake the more time up to collapse due to fire is short and the collapse mode under fire can change as respect to the pre-earthquake one
- ANALYSIS METHODOLOGY
  - Reproduction of the actual phases of the phenomena, from the application of vertical service loads and earthquake-induced damage up to the exposure of the structure to fire;
  - Identification of the seismic damage state, according to pre-fixed performance levels, by means of nonlinear pushover analyses or by non linear time-history incremental dynamic analyses;
  - Analysis under fire of structures already damaged by earthquake, starting from each

previously defined performance level;

- Correlation between the seismic performance levels and the behaviour of corresponding damaged structures under fire in terms of fire resistance and collapse mode.
- Definition of integrated seismic and fire design criteria.

- GUIDELINES FOR PREVENTION AGAINST EARTHQUAKES AND SAFETY RULES
  Development of a quantitative proposal for both fire-safety and seismic design codes, aiming at a sound design for guaranteeing fire safety of buildings exposed to post-earthquake fire risk by fitting fire resistance according to prefixed performance levels. The theoretical knowledge, empirical information and analytical capability and technology that have been developed in the community of fire protection engineers must be integrated into the structural design process.

Example of application

STEEL MOMENT RESISTANT FRAMES

Della Corte et al.,2003

- Case study: simple portal frames, multi-span multi-storey frames, made of steel
- Method of analysis
  Structural analysis under fire
  1) Dynamic time histories seismic analyses aiming at the damage identification;
  2) Fire analysis on the structural configurations distorted due to the seismic damage carried out by means of the ad-hoc software.
- Type of investigation
  1. Parametrical analysis of simple portal frames. The considered parameters are the following ratios: beam span over column height, moments of inertia of beam over column, beam over column section flexural plastic strengths, the vertical load over its elastic critical value; in addition, the extent of geometrical damage, in terms of residual drift, is taken into account;
  2. Evaluation of the fire resistance rating reduction of frames as a function of the maximum residual inter-storey drift angle and of the seismic intensity.

Faggiano et al.,2007

- Case study: simple portal frames, made of steel
- Method of analysis
  Structural analysis under fire
  1) Seismic pushover analysis under horizontal loads of structures subjected to constant vertical loads, aiming at the damage identification;
  2) Definition of the performance levels, which correlate seismic intensity and damage extent;
  3) Fire analysis on the damaged structures.
  The main difference with respect to the previous approach is the use of a numerical tool able to catch the actual phases of the fire following earthquake phenomenon in a unique model, which allows to contemporarily consider the thermal and mechanical aspects of the problem.
- Type of investigation
  1. Evaluation of the fire resistance of the study portal frames with relation to the geometrical ratio L/H, the overstrength f_y/σ_{Sd} and the S/V ratios, per unit length, of a structural member;
  2. Evaluation of the effect of the seismic-induced damage on the fire resistance and the collapse mode of the study structures.

- Results (Della Corte et al., 2003; Faggiano et al., 2007)
  - Portal frames
  Simple abaci developed for computing fire resistance rating reduction at increasing levels of residual storey drifts.
  - Multi-storey frames
  Effect of the seismic design option; effect of the structural system layout; identification of the type of collapse mechanism in fire, exhibited by both the undamaged and the earthquake-injured structures.

Pintea et al., 2008

- Case study: multi-span multi-storeys frames, made of steel
- Method of analysis:
  1) Seismic pushover analyses of the structures;
  2) Fire analysis by applying two alternative models of fire, namely the standard ISO 834 fire and natural fire scenarios (the latter being obtained considering that fire fighting measures are available in a regular fire situation, but could be partially available after the occurrence of an earthquake).
- Type of investigation:
  Evaluation of the fire resistance of frame structures, which are damaged or not by the earthquakes, depending on the seismicity of the site.
- Results:
  Effect on the fire resistance of the fire protection systems and of the structural overstrength.
  Effect on the fire resistance evaluation of the fire modelling approach.

Further developments

Further studies are needed, in order to have a deeper understanding of the phenomenon. First of all, parametrical analyses should be performed, focusing on the influence of some parameters such as the span-to-height L/H ratio, the overstrength, the massivity S/V ratio. Then, the analyses should be extended to multi-span multi-storey moment resisting steel frames, where also the influence of the fire position in the frame should be evaluated. Standard and natural fire modelling options may be used, the first one being capable of providing more general results, and the second one being able to give more realistic information, although referred to specific cases. Finally, an important objective of the research is the definition of rules and indications, based on correlations among site PGA – performance levels – fire design objectives, in a PBD approach, considering different fire scenarios.

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COST C26 – WG1

Datasheet no. 15

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ROBUSTNESS UNDER FIRE

Description

- Robustness is concerned with the control of failure of a system disproportionate to the initiating exceptional event. It is important to distinguish fire as an exceptional event from others, e.g. impact, explosion, see below. It is also important to understand the additional requirement on robustness under fire to the usual requirement of preventing progressive structural collapse under fire.
- Fire loading is already being included in building design as an accidental action in EN 1990-1:2002. Therefore, design for fire loading is not design for exceptional loading. Fire loading becomes an exceptional loading condition when the actual fire condition is much more severe than the design fire scenario, e.g. standard fire resistance or other nominal fire condition such as the parametric fire curve in EN 1991-1-2:2003; or the performance of real fire protection measures are not as well as assumed, e.g. degradation in or faulty application of structural fire protection materials, use of inadequate/sub-standard structural fire protection materials; or design faults.
- Robustness in fire should also address the issue of fire spread, in addition to control of progressive structural collapse. Control of fire spread is nominally included in "integrity" under fire resistance.

Field of Application

- Design for robustness under fire is an important issue that still requires a substantial amount of research effort from the structural and fire research community. Therefore, this technical sheet will be different from other technical sheets in this series. Instead of being able to give some design guidance and well-established solutions as in other technical sheets, this technical sheet will mainly pose some questions and suggest possible ways of tackling them. It is intended that this technical sheet should help the reader to formulate their questions when dealing with robustness in fire This technical sheet will also give some general advice on improving structural robustness under fire.
- By their nature, it will not be possible to precisely quantify the extent and magnitude of any exceptional loading. Therefore, a risk based assessment approach should be taken when considering a particular building under fire attack, in which the importance of the building should be a major consideration. As a first approximation, the building classification system in EN 1990-1:2002 may be adopted. It is often not possible to carry out the risk assessment with good precision, therefore, sensible engineering judgement should be exercised.
- Combined aeroplane impact and fire exposure caused collapse of the World Trade Center buildings on September 11, 2001 (McAlister et al 2002). However, it is unlikely that such an exceptional loading situation would be a feasible design requirement for the majority of buildings for which design for robustness is an important consideration. Therefore, this technical sheet is primarily concerned with the accidental fire situation in which fire is the sole cause of damage to the building. Nevertheless, many of the issues and suggestions in this technical sheet may still be applicable to other exceptional

loading situations involving fire attack. Due to the limited availability of data, this technical sheet will be mainly based on studies related to steel structures.

Technical information

- As introduced in the opening section ("Description") of this technical datasheet, consideration of robustness under fire should address two issues: control of progressive structural collapse and control of fire spread through the building. To present issues involved in progressive structural collapse, this technical datasheet will use a framed structure to illustrate a few feasible consequences of accidental fire loading and suggest possible ways of improving structural robustness under fire attack.
- Consider a two-bay multi-storey framed structure under fire attack on the ground floor as shown in Figure 1. Accidental fire loading may cause local failure of the structure in four possible ways as shown in Figures 1(a) – 1(f).

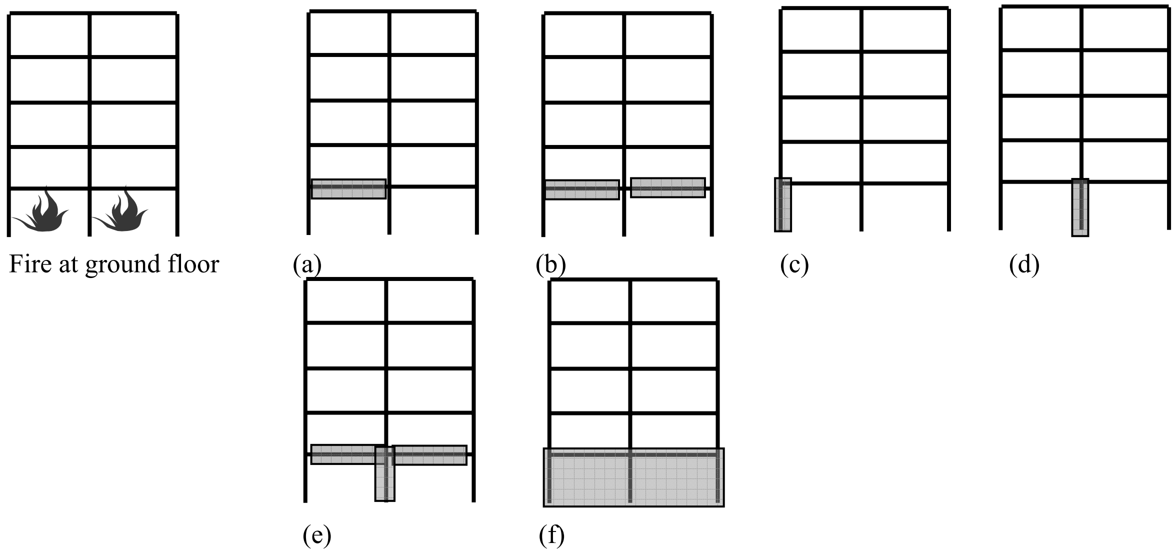

Figure 1. Possible fire damages to the structure due to the fire

- In the case of the exceptional fire causing failure of one beam as in Figure 1(a), it is possible to control the extent of further structural damage by allowing the beam to develop catenary action to maintain integrity of the structure. The column design conditions are likely to be the same as in the original design. To enable catenary action to develop, the beam to column joints should have sufficient strength to resist the catenary forces and sufficient ductility to accommodate the large rotations of the beams. In the case of failure of two adjacent beams as shown in Figure 1(b), the effect of catenary action in the beams may still be exploited and again the joints should possess sufficient resistance to the catenary force in the beams as well as ductility. In addition, it is also important to recognise that the column between the two failed beams may have a buckling length that extends to two floor heights.
- If an edge column fails, the consequence of this column failure may depend on the frame joint properties as well as load bearing capacity of the floor slabs. If the joints are rigid, it may be possible for the damaged framework to continue providing load bearing capacity to control progressive collapse. If the joints have minimum bending moment resistance, regardless of the joint resistance to tension (tying resistance), the structure may suffer progressive collapse (Wang and Orton 2006). One means of controlling progressive collapse is to make sure that the connected floor slabs have sufficient bending resistance to bridge over the failed column. In the case of failure of an interior column as shown in Figure 1(d), it is possible to prevent progressive collapse by

increasing the bending moment capacity of the connected beams and joints or by exploiting catenary action in the beams. Again joint resistance to catenary forces and ductility should be ensured.

- When the internal members in the fire enclosure fail as shown in Figure 1(e) (e.g. the edge members are protected by additional, but unaccounted for construction such as walls), it is possible to control progressive collapse through the development of catenary action in the beams. Again, issues such as joint resistance to catenary forces and ductility, increased column buckling length, should be considered. If failure involves all the structural members in the fire enclosure as shown in Figure 1(f), there is a great potential of progressive collapse of the structure, unless additional procedures are incorporated. For example, it may be necessary to include a floor height truss so that the loads from the failed structural members can be re-distributed by the floor height truss via tension forces in the columns below.
- Now consider the issues related to control of fire spread. Although fire resistant design for integrity is aimed to ensure containment of fire attack within the initial fire enclosure, it is still possible for extensive fire spread to occur. Failure of the Winsor building in Madrid is sufficient to demonstrate this point (Callavera et al. 2005 and Fletcher et al., 2007). At present, satisfaction of fire integrity is based on standard fire resistance test of individual components. This may not be sufficient. To prevent excessive fire spread, two additional measures will be needed: (1) to ensure that the various building components can withstand the attack of real fires; (2) to ensure that the various building components are properly assembled together, with sufficient redundancy to ensure fire integrity of the building in case of some component failure. Although it is recognised that neither of this can be easily undertaken, their consideration at the design stage should result in a building with better fire integrity than simply following prescriptive rules on fire integrity, which is the current practice.
- Structural fire engineering has developed rapidly and it is now possible to use sophisticated finite element modelling techniques in fire engineering design of structures. Often, structures designed by using such advanced techniques can only satisfy fire resistance requirements based on structural performance at very large deformations and rotations. Whilst such structures may be structurally robust, i.e. very low risk of progressive structural collapse, the problem of excessive fire spread may arise. For example, large rotations in concrete floors near the supports may result in cracking and consequent development of cracks through the depth of the floor slabs, which may cause fire spread through the cracks. At present, modelling this phenomenon still represents considerable challenge to the research community. Nevertheless, the fire engineering profession should incorporate measures to mitigate the consequences.

Structural aspects

- Although vigorous methods of satisfying the requirements of structural robustness under fire are still not available, it is easy to identify the following two critical aspects of structural behaviour in fire: (1) variations of forces in the structure (Sokol and Wald 2007), particularly in the joints, in fire; (2) realistic resistance of joints in fire, in particular their resistance to tension forces in the connected beams. The associated technical sheet No. 5 presents detailed information on quantification of joint behaviour in fire. Therefore, this technical sheet will only present a review of some research studies on joint performance in fire with specific reference to structural robustness.

- When a structure is exposed to fire, the structural materials lose their strength and stiffness at high temperatures. Accompanying this is the effect of thermal elongation and large deformations. All these will cause the internal forces in the structure to change significantly during the course of a fire attack (Wang 2002). For example, for ambient temperature design, the forces in a beam-column joint is either shear or combined shear and bending moment. There is very little axial force. Under fire conditions, the bending moment may change. More importantly, there may be significant axial forces, being compressive due to restrained thermal elongation of the connected beam being restrained, or tensile if the adjacent beam develops very large deformations and starts to pull-in under catenary action. Tensile forces may also develop in the connected beams in cooling when contraction of the beam is restrained. Some recent numerical and experimental results have become available to help quantification of the variation in internal forces in the joints. For example, Yin and Wang (2004) used the finite element package ABAQUS and analysed the development in axial force in steel beams with various levels of axial and rotational restraints at ends. As discussed in the "Technical Information" of this technical sheet, catenary action may be used as a means of controlling disproportionate collapse. The theoretical study of Yin and Wang (2004) indicates that at this stage, the most important parameter to consider is the axial restraint stiffness to the beam at the ends, e.g. joints. Other features, such as joint rotational restraint, temperature distribution and behaviour of the beam (whether or not lateral torsional buckling occurs) have very little influence.

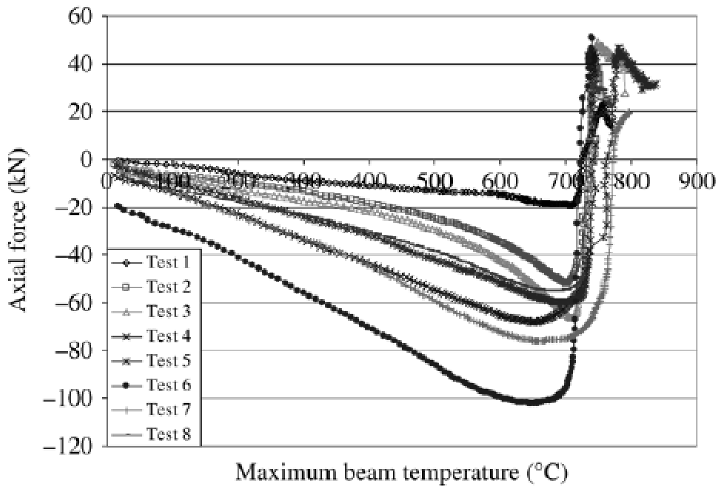

Figure 2. Axial forces in beams

- A separate theoretical study by Yin and Wang (2006) suggests that catenary action occurs when the bending moment resistance of the beam has been exceeded. For solid steel beams, this happens when the temperature in the steel beam has reached the conventional limiting temperature without consideration of axial restraint. However, for steel beams with openings, catenary action may occur at a much lower temperatures due to large compressive force in the steel beam, which may cause the steel structure around the openings to experience premature buckling and hence to induce catenary action. This is important when considering robustness of structures using beams with openings, which will include cellular beams and trusses.
- Ding and Wang (2007) reported experimental results obtained from fire tests on beam-column assemblies with different types of joints between steel beams and concrete filled tubular columns, including two tests in which cooling behaviour of the beam was studied. Figure 2 presents the variation of axial forces (tension being positive) in the test beams as a function of the maximum temperature in the beam. The change of axial force in the beam from compression to tension is rapid, occurring when the beam was experiencing rapid increase in lateral deflection. There was no sign of collapse of the test structure when the beam was in compression. Failure (joint fracture) occurred when the beam was in catenary action. Joint fracture has been observed in real fires. However, this

has been often associated with cooling behaviour. The test results of Ding and Wang (2007) clearly suggests that joint fracture can happen in heating. This is important because in an exceptional fire situation (not the normal fire design scenario), catenary action may occur rather early in fire after which the structure may be subjected to a prolonged heating period. If stringent control of disproportionate collapse in fire is necessary (depending on the importance of building), then it is important that the joints should be of a "robust" nature. Whilst at this stage, it is still difficult to define what makes a robust joint, it is unlikely that simple fin plate joints or web cleat joints will be classified as robust joints. Extended end plate joints are likely to be necessary. For structures using tubular or concrete filled tubular columns, making the conventional type of end plate joints would necessitate the use of blind bolts. This is unlikely to be adequate since the blind bolts have mainly been developed to resist shear and their pull-out strength under tension in the connected beam may not be sufficient. To solve this problem, the so-called reverse channel joints may be used, in which a channel section is welded to the steel tube and the steel beam with end plate is then attached to the web of the channel as in beam to open column joints. An example (Ding and Wang 2007) is shown in Fig. 3.

Figure 3: The reverse channel joint tested

- Methods of improving robustness of joints (hence robustness of structures) in fire include using composite joints with continuous reinforcement across the joint and embedding elongated extended end plate joints in concrete.

Guidelines

- At present, design for robustness is commonly carried out according to prescriptive rules. The most important aspect of the prescriptive approach is to provide sufficient tying resistance. The tie force apprach, which predicts the horizontal forces needed to ensure the integrity of a structure under exceptional loading conditions, mostly by explosion, was based on large panel buildings and the best engineering judgement after the Ronan Point accident, see Advisory Desk. The criteria to be satisfied, e.g. two M16 bolts in tension are sufficient for the structural integrity, were replaced by the simple estimation based on the span of the beams, its spacing and loading, see Advisory Desk 1993, Way 2005 and Owens 1992. According to Annex A6 of Eurocode EN 1991-1-7: 2006, the required tie forces may be calculated from

$$T_i = \min\left[k\left(g_k + \psi\, q_k\right) s\, L;\ 75\ \text{kN}\right] \qquad (1)$$

  where k is the transformation factor; for internal ties k = 0,8; for perimeter ties k = 0,4, g_k is the characteristic value of permanent action, is the combination factor according to the accidental load combination, q_k is the characteristic value of variable action, s is spacing, and L is the span of the tie.
- It should be appreciated that the existing tie force approach is an extremely rudimentary approach and has little scientific basis. Although it has served society reasonably well (e.g. very low frequency of disproportionate collapse since the introduction of this approach), it is expected that the recently started intensive studies on structural robustness will lead to better understanding and more rational design methods.

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COST C26 – WG1

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FIRE DAMAGED STRUCTURES

Description

- Assessment of fire damages to structures follows a similar general process as appraisal of existing structures.
- It is possible to restore a fire damaged structure to its original load carrying capacity.
- In making a decision about repairing a fire damaged building, considerations should be given to aesthetic appearance, the reliability of repairs, the views of insurance company and the client, in addition to technical feasibility.

Field of Application

- This datasheet is concerned with technical aspects of appraisal of fire damaged steel, concrete and stone structures.
- Assessment of a fire damaged structure differs from fire resistant design of the structure. In fire resistant design of a structure, the design fire is assumed; the material properties are those at high temperatures and the structure is assessed for reduced structural loads under the fire limit state. In assessment of a fire damaged structure, the engineer has to take into consideration the actual fire that has occurred in the structure; the material properties are those at ambient temperature but after being exposed to high temperatures; the repaired structure should be able to resist loads corresponding to the ultimate limit state, including the additional weight of any repair materials.
- In fire resistant design of a structure, the engineer obtains data by making suitable assumptions. In assessment of a fire damaged structure, the engineer obtains data by gathering evidences related to the specific damaged structure and the actual fire. It is important that the appraisal process starts as soon as the building can be safely entered and before the removal of debris to preserve vital evidence.

Technical information

- The general procedure of appraisal of a fire damaged structure comprises of the following steps: initial site visit, desk study, detailed collection of evidence, damage assessment and specification of repairs.
- The purpose of the site visit is to gain an early indication of the scale of damage to the structure and to advise on safety of the building and to recommend measures to protect the general public and other essential personnel.
- The purpose of the desk study is to collect relevant information (e.g. original design of the building, construction materials, usage before fire, cause of fire, duration of fire, fire spread, contents left unburnt) by examination of physical evidence, interview of the fire brigade and witness. Using the preliminary data gathered, the engineer should establish a strategy for more detailed assessment and data gathering.
- Fire damages to a structure can be broadly grouped into four categories: no

damage/superficial damage, total damage, major damage and reparable damage. No/superficial damage requires no structural repair; total damage leads to scraping of the total structure; major damage requires replacement of the damaged structural members. For these categories, decisions can be made quickly without the need to undertake detailed assessment. Reparable damages are those that may be repaired but there is a high degree of uncertainty about the residual load carrying capacity of the structure. The main objective of damage assessment is to decide with as much confidence as possible the residual mechanical properties of the fire damaged materials so that the fire damaged structure can be restored to its required load carrying capacity.

Structural aspects

- The residual mechanical properties of fire damaged materials may be obtained using the following methods: (1) by direct measurement using Non-Destructive Testing (NDT) and destructive testing; destructive testing should be kept to minimum and should only be used when there is low confidence in NDT results; (2) by direct assessment of maximum material temperatures and link to material residual mechanical properties – temperature relationships; (3) by establishing the fire history, from which the material temperature history may be established using heat transfer; afterwards, using the residual mechanical properties – temperature relationships. Due to uncertainty in results obtained from these different methods, it is important to correlate the different results to improve confidence in them. It is also important to make conservative (safe) assumptions when evaluating residual load carrying capacities of fire damaged structures; for example, assuming simple supports and ignoring any beneficial effects of the restraints.
- The documents (Kirby et al 1988, Concrete Society 1990, ISE 1996) described in the **Guidelines** section of this technical sheet may be consulted to obtain residual mechanical properties – temperature relationships for steel and concrete. Additional data may be obtained from Outinen and Mäkeläinen (2004) for steel; from Dias (1992) for concrete and from Yan and Wong (2007) for high strength concrete; and from Hajpál and Török (2004), Hajpál (2008) and Török and Hajpál (2005) for sandstones.
- Materials expand at high temperatures, which may cause brittle materials remote from the fire site to suffer damage. It is important to assess the entire structure for fire damage. For example, expansion of floors directly involved in a fire may damage the walls in remote places from the fire.
- NDT methods for fire damaged steel include Hardness test and metallurgical microscope. Hardness test is simple and easy, but the hardness test results should not be used to guarantee the material to an appropriate specification, for which coupon tests are required. Microscopic test requires specialist personnel and equipment. It is used only when it is essential, e.g. to provide information on the micro-structure of metal so as to establish an accurate picture of the heating environment.
- Methods of assessing fire damage to concrete include colour observation (e.g. pink indicating about 300^{o}C), visual classification, NDT testing (Schmidt hammer, ultrasonic pulse velocity, thermoluminescence) and destructive testing (cores). It is important to choose the appropriate testing method before detailed assessment starts.
- Steel recovers much of its initial strength and stiffness after fire exposure. Therefore, a fire damaged steel structure can normally be reinstated. Unless severely distorted to affect appearance, steel structural members can normally be retained. High strength

bolts are made by quenching. Exposure to high temperatures above 500^{o}C has the similar effect as tempering, which would reduce the residual strength of bolts. Generally, bolts after exposure to high temperatures should be replaced. If the reinstated steel structure requires fire protection, it is important that smoke deposits on the steel surface are removed before application of fire protection materials.

- Various methods may be employed to repair fire damaged concrete structures, including reconstruction (major repair after extensive damage or sprayed concrete is difficult), sprayed concrete, resin repairs (for repairs to lightly spalled areas), overcladding (non-structural materials such as plasterboard, to restore appearance/restore fire resistance/durability), provision of alternative supports.

Guidelines

- Reference Kirby et al (1986) provides guidance on reinstatement of fire damaged steel and iron framed structures. It gives detailed residual mechanical properties of different types of structural and reinforcing steels, iron and bolts after exposure to different high temperatures, metallurgical evaluation of fire damaged structural steelwork and a number of case studies. Figure 1 is a flow chart for reinstatement of fire damaged steel structures.
- Reference Concrete Society (1990) provides detailed guidance on assessment of fire damaged concrete structures and design for repair. Detailed information of the effects of high temperatures on structural materials is provided. Different popular methods of assessing fire damaged concrete are described. Detailed guidance is given on how to design and specify repair methods to restore the load carrying capacity of the fire damaged building. A number of detailed examples are included to demonstrate application of the procedures in this document. Figure 2 is a flow chart for assessment and repair of concrete structures.
- Reference CIRIA (1986) provides more detailed explanation of different methods of assessing concrete, some of which are referred to in Reference Concrete Society (1990). This reference is for general use of assessing concrete, but many of the test methods are applicable to fire damaged concrete.
- Reference ISE (1996) is a general document for appraisal of existing structures, the general procedure of which may be followed in assessment of fire damaged structures. It also provides information on temperature effects on a selection of non-structural materials, which may be used to establish the history of the fire.
- Reference SCIF (1991) provides a detailed case study of assessment of fire damage to the Broadgate building (a steel framed composite structure) in London, which was extensively damaged by a severe fire during its construction before fire protection to the steelwork was installed. The fire damaged structure was successfully reinstated by replacing the fire damaged floors and columns. Good behaviour of the unprotected Broadgate building under a severe fire was the initial impetus to the much publicised Cardington structural fire research test programme, which formed the backbone of many of the structural fire engineering research studies of the past 15 years or so.

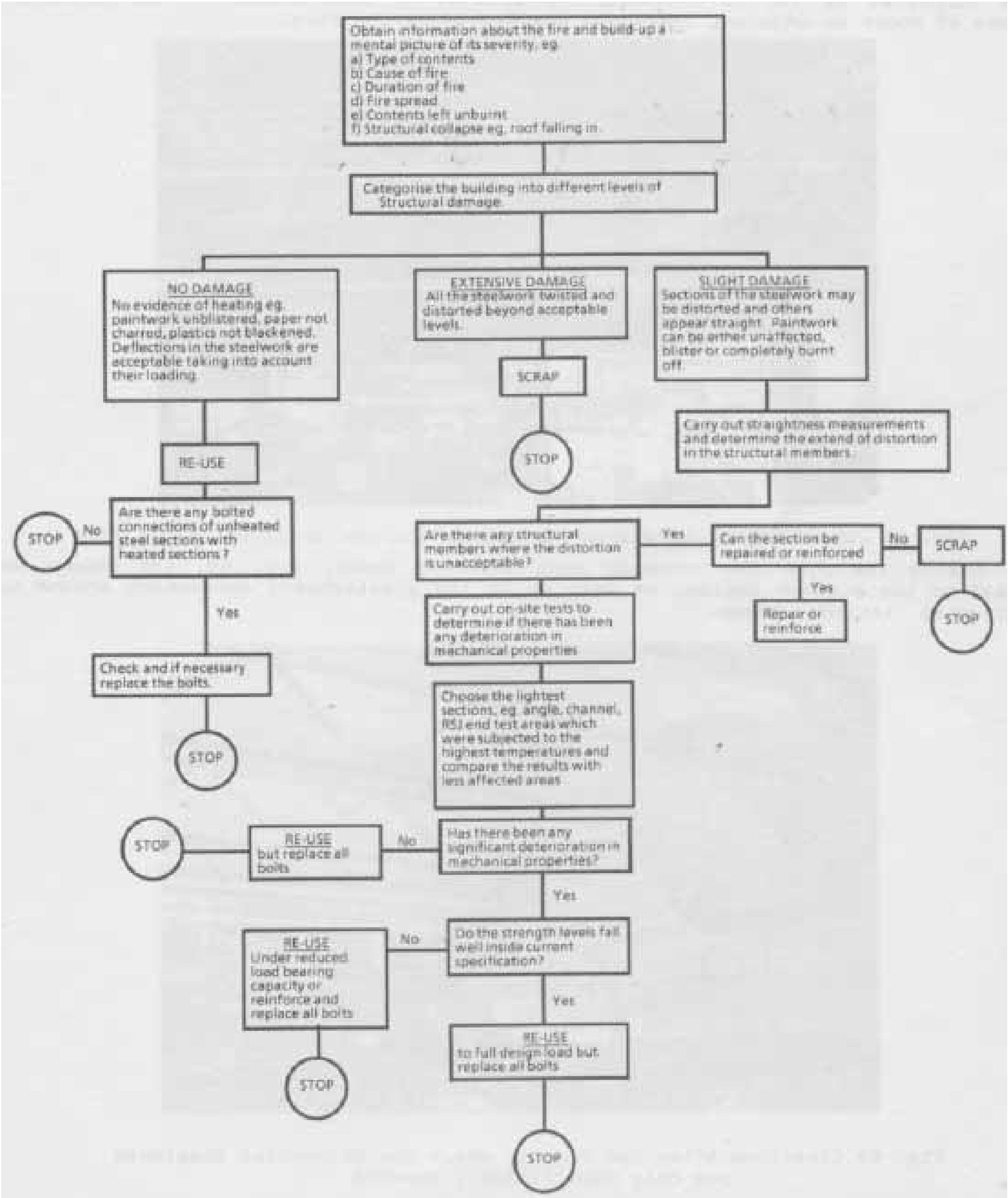

Figure 1. Appraisal procedure for fire damaged steel structures (Kirby et al 1986)

- A study of sandstones at elevated temperatures shows that heating affects the internal structure and mineral composition of natural stones, which influence the petrophysical parameters (porosity, strength, water adsorption, colour) of the stones. These changes are not always adverse. Hajpál & Török 2004 and Török et al 2005 describe how the mineralogical composition and texture of natural stones influence their resistance to fire and thermal characteristics. The heat resistance of different quartz sandstones depends on the type of the cementing mineral, the amount of cement (grain/cement ratio), the grain size (fine, medium, coarse) and the grain to grain or matrix to grain contacts. Compact stones show more dramatic change in porosity at elevated temperatures than the less cemented ones. A porous and cement rich stone is more adaptable, being able to accommodate thermal expansion induced additional stresses. Silica cemented, ferruginous or clayey stones are less sensitive than the carbonatic ones, which disintegrate at higher temperatures.

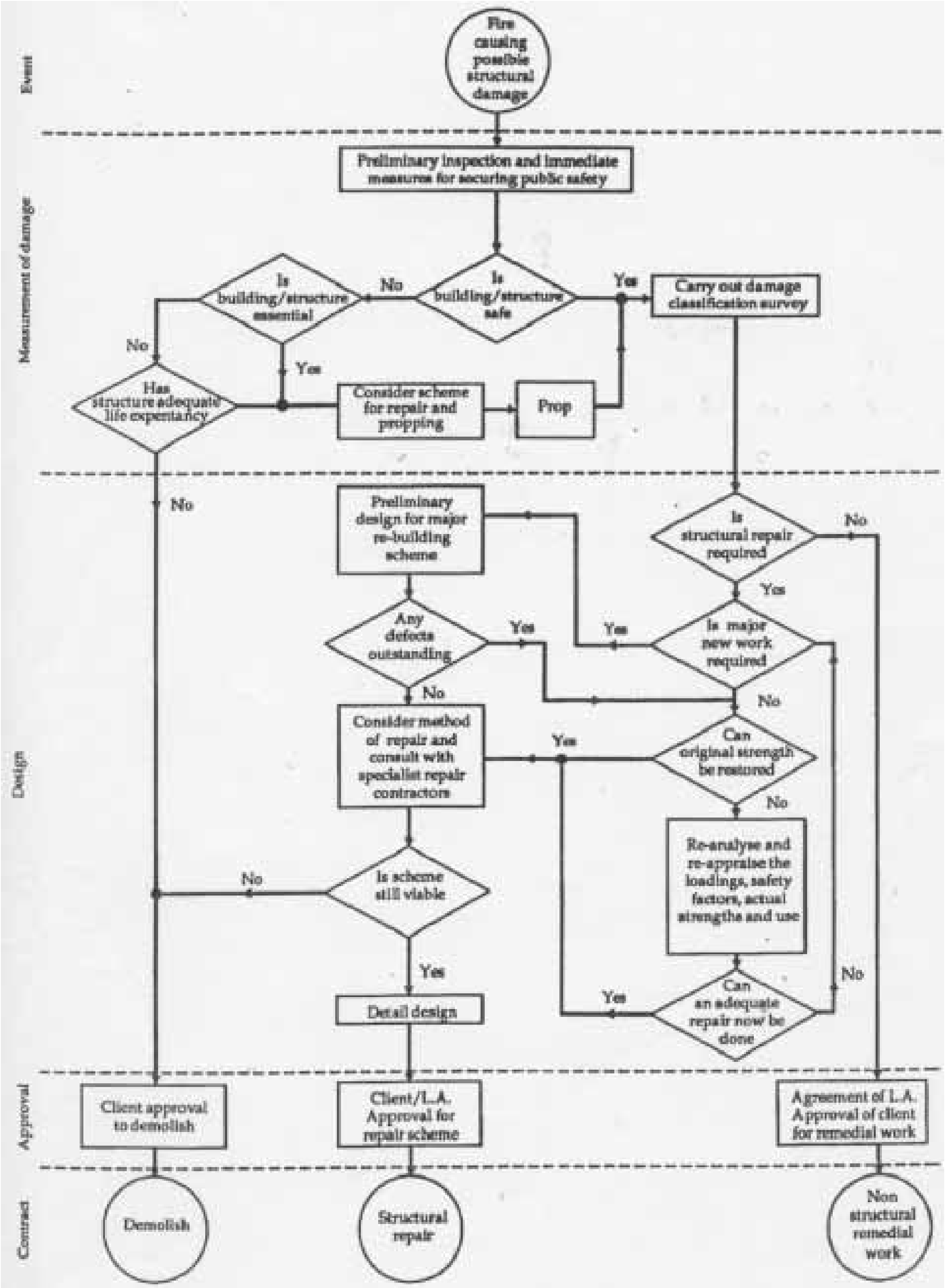

Figure 2. Appraisal procedure for fire damaged concrete structures (Concrete Society 1990)

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