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Fire Safety in Buildings

A Western Balkan approach and practise

Editors
Dr Mirjana Laban, Dr Igor Džolev,
Dr Mirjana Malešev, Dr Vlastimir Radonjanin

eBOOK

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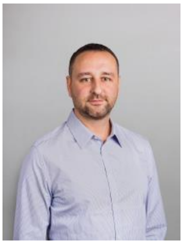

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PREFACE

Fires in buildings are now bigger, more unpredictable and more dangerous than ever before. The lately fire events in high rise buildings and renovated buildings indicated the current regulatory requirements around fire safety in buildings are not providing an adequate level of fire protection for residents.

The way in which fire safety statistics are recorded across Europe varies greatly. Currently, there are no European-wide standards for collecting or analysing data on fires, which makes it difficult to compare national statistics on fire casualties or fire damage. Available fire safety statistics show considerable improvements over the last decades, but structural fires, especially residential fires, remain a critical concern. Also, these statistics provides an obvious indication of the dominance of dwelling fires and the importance of action on domestic fire safety.

Fire safety of buildings depends on various elements and therefore requires a holistic approach that addresses both preventive and constructive fire protection. Preventive fire protection focuses on avoiding fires, whereas constructive fire protection includes fire performance of materials and systems solutions for the building and its envelope. In that regard, fire-safe buildings need construction materials and products to be approved, installed and maintained responsibly and in accordance with all regulations.

We need to build sustainable, and for fire safety, this means building fire resilient constructions. Only buildings which are able to resist to, adapt to and recover from a fire and resume to their essential functions in a timely and efficient manner, can foster a truly sustainable future.

This book is the result of K-FORCE Erasmus + project, aimed to establish and improve higher and life-long education in the field of Disaster Risk Management and Fire Safety in the Western Balkans.

We hope that this book will contribute to better understanding the complexity of fire problems in buildings and will be the useful literature for fire safety students and professionals.

The Editors

Acknowledgements

The editors of this book would like to express their appreciation to the Knowledge FOr Resilient soCiEty (K-FORCE) project partners, for their assistance and support. We are also grateful to the authors who contributed their research work to this publication.

Section I

Fire, Materials and Structures

FIRE SAFETY ENGINEERING

Edisa Nukić, Jelena Marković

1. INTRODUCTION

When designing and constructing a building, fire protection must be provided; it should be integrated into design process from the very beginning. Since fire occurrence very often relates to the failure to meet adequate preventive measures through planning, design and construction, it is important that everyone involved in building design - architects, engineers, interior designers - are aware of fire safety issues at each stage of the process. Some elements that are necessary for fire protection, which are not designed during construction planning phase, are difficult to incorporate later and are often extremely expensive.

Fire safety in buildings is determined by factors such as escape, evacuation and rescue in the event of a fire, the ability of the building to resist fire and to limit fire and smoke spread (within the building as well as on adjacent facilities), and ensuring fire departments access for efficient rescue and firefighting actions.

At the very beginning of the firefighting and legislation in this field, insurance companies had an important role to play in protecting the property, but not the safety of people's lives. Today, besides the primary goals of fire protection - the protection of life and property, there is another important goal - maintaining the function. It is clear that, regardless of the damage caused by fire, people must continue to perform their life functions.

For *life safety*, it is necessary to create conditions for safe evacuation and escape. To achieve this goal, it is necessary to have a timely warning system, adequate evacuation routes, and ensure that evacuees are not exposed to smoke during evacuation. In some buildings, it is necessary to plan special measures and solutions for the presence of occupants who cannot save themselves, such as, for example, hospitals with immobile patients, institutions for the care of old and handicapped persons.

Property protection is achieved by protecting the construction structure, elements and contents of the building, as well as by fire protection from adjacent buildings. The property protection also includes protection of intangible assets such as business interruption or unrecoverable loss of cultural heritage.

In achieving the goal of *environmental protection*, the primary concern are gas pollutants emissions from smoke and liquid pollutants generated by the use of fire extinguishing agents since they can have a significant impact on the environment. An efficient way of limiting these impacts is fire extinguishing in the initial phase, which can be most easily achieved in buildings by installing fixed fire extinguishing systems.

The complexity of interaction between people, buildings and fires is such that no single approach or calculation method can be applied to all types of buildings in different circumstances. Therefore, fire protection engineering requires from designer great level of engagement, responsibility and experience rather than a mere application of the prescribed codes. The concept of fire safety engineering is applicable wherever there is a fire risk, where it is necessary to quantify the risks and provide an optimal solution for preventive or protective measures (so-called active and passive measures) application. In case of fire, a good building design and its maintenance are crucial in enabling safe evacuation, as well as limiting fire spread to the minimum possible extent and ensuring safe access to firefighters and rescuers. The best fire protection solutions take into account specific design of each building and the way its occupants use it, combining active and passive measures in a holistic approach (Figure 1).

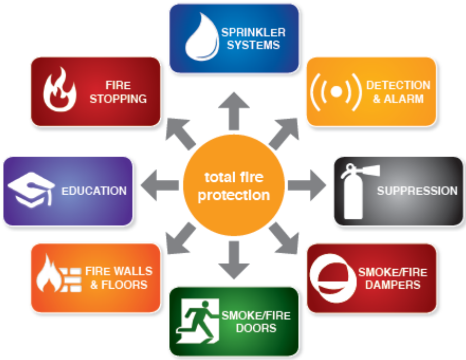

Figure 1. Holistic approach to fire protection (www.lifesafetyservices.com).

1.1. Fire safety design

Fire is one of the greatest dangers for life and property in buildings. Material damages and human casualties caused by fire are not negligible, as well as average number of fires that occur annually. Statistics show that the majority of fires are in residential and public buildings. Due to fire in Europe an average 8 people (of million) lose their lives every year, while a larger number gets hospitalized. According to the World Fire Statistics Centre in Geneva, fires damage the world economy by about 1% of gross national income each year. Since fires are very difficult to prevent, the issue of this risk is intensively dealt with by state governments that continually improve their fire protection strategies. Therefore, a large number of national and regional regulations on fire protection have been established specifically for buildings.

It is difficult to give a precise definition of „fire-safety engineering“. The following should be acceptable: the application of scientific achievements and engineering principles to the effects of fire in order to reduce human suffering and damage to property through quantification of risks and to provide optimal solutions by implementing preventive and protective measures [3].

More precisely, the basic objective of fire safety engineering is to reduce the risk of death, injury, material damage and environmental pollution to an acceptable level.
Although fire safety engineers develop and deliver fire safety design solutions, designers from other areas are often required to make a major contribution to developing a fire protection strategy.

Three are the main approaches in fire safety engineering:

- *Comparative approach* that proves that the project provides a level of safety equal to that which would have been obtained by applying the prescribed codes.
- *Deterministic approach* aimed at showing that set of defined conditions will not occur, which is determined by the initial assumptions (most often the worst possible

case). When there is any doubt about the reliability of input data, a conservative approach has to be adopted. This may require the use of exact safety factors to compensate for ambiguities in assumptions.

- *Probabilistic approach* aimed at demonstrating that the probability of a given event that has occurred is acceptable. This is usually expressed in terms of annual probability of occurrence of an unwanted event (for example, the probability of individual deaths due to a fire of 10^{-6} or one per million).

Fire safety engineering concept provides a framework that allows designers to demonstrate that the functional requirements of the legislation have been met or improved, although the adopted solutions for the project do not fall within the prescribed codes recommendations and guidelines. In addition, it also provides functional objectives, besides the life safety, related to: property protection, business continuity, environmental protection goals and sustainability. When there is a greater difference between building design and guidance offered by codes, then analytical techniques that analyse fire growth, smoke control and human evacuation can be required to prove the overall fire safety strategy. The first step in the preparation of such an analysis is building geometry definition, functional planning of building materials and general use of the building.

Fire safety engineering may be the only sustainable way to achieve satisfactory fire safety standards in some large and complex buildings, and often the most effective way to adapt existing buildings and those of cultural value. It can be usefully applied to certain building project elements, while the rest of the building will be designed according to the prescribed codes.

The application of fire safety engineering makes it possible to recognize useful effects. For example, providing an automated fire suppression system can reduce the projected fire size, which in turn can lead to a more cost-effective project of a smoke control system or a reduced range of fire protection for the building.

The elements of fire safety engineering can be easily identified, and they relate to the safety of lives and property. These areas are not mutually exclusive, as an activity that increases life safety can increase the property safety as well.

The basic aspects of fire safety engineering are:

- Ignition and combustion control - control of material flammability within the structure, fire growth control or fire protection control - smoking ban or open flame use.
- Evacuation control - by imposing legal requirements for the provision of adequate escape ways or by educating occupants.
- Fire detection and control - installation of fire detection devices (manual and automatic systems); fire smoke control; fire extinguishing systems.
- Fire spread control in building or on surrounding property - passive protective measures (such as fire compartments) or control (by determining safe) distances between buildings or by mechanical means (such as ventilation, smoke curtains or sprinklers).
- Preventing construction collapse in a fire - structural fire protection (preventing fire transfer through construction elements and/or structures); protection of structural elements and/or structures.

1.2. Ignition control

1.2.1. Introduction

Construction and fire protection laws clearly define essential requirements for buildings that are to be provided in design and construction regarding fire protection. Buildings must be designed and constructed so that in the event of a fire:

- maintain the bearing capacity of the structure over a certain time,
- the occurrence and spread of fire and smoke inside the building is limited,
- fire spread to adjacent buildings is limited,
- allow occupants to leave the building unharmed, or to allow their rescue, and
- provide the rescue services safety.

Each of these requirements requires the implementation of certain construction measures:

- protection of building load-bearing structures against the effects of fire,
- division of buildings into smaller spatial units (fire compartments) with adequate fire resistance,
- division of the building exit roads by the construction structures,
- construction of firefighting access around the building.

These requirements, apart from regulations defining fire protection elements of a building, can also be addressed by the engineering methods as defined in point 2.3 of the fundamental European document in the fire protection field.

European Construction Products Regulation CPR 305/2011 defines in Annex I, that the construction works and the building as a whole, as well as their individual parts, must correspond to the intended use, taking into account in particular the health and safety of persons involved in the entire cycle of construction and exploitation of buildings and works. The first requirement is mechanical resistance and building stability and the second requirement relates to fire safety, while other requirements do not apply to the design aspects. In fire safety requirements, among other requirements, it is stated that the building must be designed and constructed in such a way that, in the event of a fire, it must preserve the structure load capacity for a certain period of time.

With proper selection of construction materials, construction elements and constructions with regard to their fire resistance and proper building design the fundamental requirements for fire protection are fulfilled. In addition to the undertaken construction measures, it is very important to educate and inform people about possible causes and places of fire. Proper handling, knowledge and available technical means are helping to eliminate or reduce the fire hazard.

Fire development in building is influenced by the *quantity and characteristics of flammable materials and the availability of oxygen*. If a fire is indoors, such as a building, it will have a huge impact on the fire behaviour because heat and smoke cannot be freely expanded. Also, the expansion of smoke inside the building is one of the important aspects of design, as in practice smoke is often decisive for the successful evacuation of people from the facility. Even buildings with construction made of non-flammable materials contain, without exception, the materials that are burning under certain circumstances. On the other hand, on the basis of the tests, flammable materials can have negligible significance.

Figure 2 shows fire development phases with the corresponding fire risks such as: flammability, flame spread, temperature increase, smoke, structure destruction and the like.

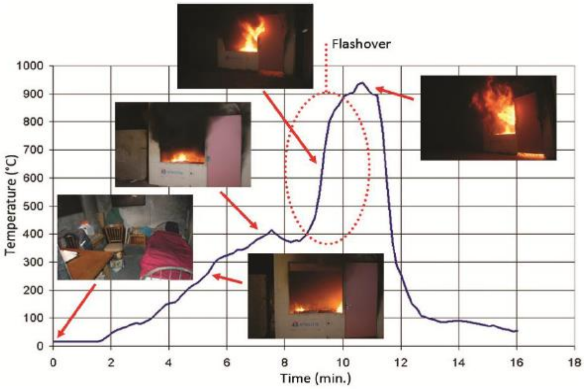

Figure 2. Fire development and flashover occurrence in the room [10]

Smoke usually contains unconsumed pyrolytic gases and its temperature increases due to the heat release. At a certain point, when the radiation level is about 20 kW/m^2 and the smoke layer temperature is around 400-600° C, all flammable gasses in the room begin to burn in a short time. This moment is called flashover when fire intensity is greatest.

According to NFPA 921-2004, *flashover* is a transitional phase of indoor fires development where surfaces exposed to thermal radiation reach the ignition temperature and fire spreads at high speed through the space as a result of it fire affects the entire enclosed space. Flashover occurs when the layer of glowing gas reaches the level of radiation energy (current) to the unlit objects of about 15-20 kW/m^2. This level is usually sufficient to ignite ordinary flammable materials, and corresponds to temperatures of at least 500-600 °C (932-1112 °F). In each building, the goal must be to limit fire spread as much as possible to allow safe evacuation of people and the rescue teams operations.

Before the flashover, fire can be controlled by active fire suppression measures such as a sprinkler and hand-held fire extinguishers, Figure 3. After the flashover, these active fire-suppression measures can no longer help. The amount of heat released is simply too large for efficient fire suppression. Passive protection measures, such as compartmentalization and structural fire protection, become important for preventing or limiting the fracture of construction or fire spread.

Depending on the building characteristics, fire load and measures and guidelines for people and property protection, a scenario for fire development in building can be developed. Figure 3 shows curves of real and standard indoor fires, where active protection measures that have the role of preventing fires and smoke spreading are visible.

One of the fundamental requirements of fire protection is proper choice of building materials according to their behaviour in fire. The fire characteristics of construction materials that can determine the speed of fire spread, type and quantity of combustion products, and its fire behaviour are related to: fuel, flammability, flame propagation speed, burning droplets, smoke production, toxic gases and thermal power. Proper application of construction materials depends on the knowledge of these characteristics.

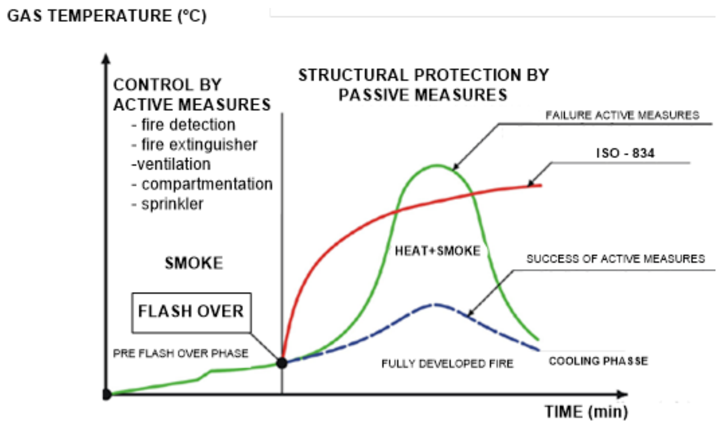

Figure 3. The various phases during the development of a natural fire [6]

European Standard EN 13501-1 [14] provides the reaction to fire classification procedure for all products and building elements, Table 1.

Materials are classified in non-combustible building materials (euro classes A1 and A2) and combustible construction materials (euro class B, C, D, E and F) where the determining factor in classification is the time at which the flaming spread.

Table 1

Classification according to European Standard EN 13501-1

	Classification of materials in reaction to fire	
	A non-combustible materials	
A1	does not contribute to fire	there is no potential for blasting the flame
A2	does not contribute to fire	there is no potential for blasting the flame
	B combustible materials	
B	very limited contribution to fire	there is no potential for blasting the flame
C	limited contribution to fire	possible blasting of the flame
D	acceptable contribution to fire	possible blasting of the flame
E	acceptable behaviour in fire	possible blasting of the flame
F	the property (in relation to the burning of the flame) is not determinable	possible blasting of the flame
	Additional class – "s" smoke emission level	
s_1	Very limited amount of smoke developed	
s_2	Limited amount of smoke developed	
s_3	No demands to the amount of smoke developed	
	Additional class – "d" flaming droplets and/or particles production	
d_0	No burning droplets or particles	
d_1	Limited amount of burning droplets or particles	
d_2	No demand to the amount of burning droplets or particles	

Table 2 shows the comparison of material classification with respect to the fire contribution according to German DIN 4102 [13] and European EN 13501-1:2010 [14].

Table 2
Classification of materials to combustible

Contribution to fire	Classification of materials		Material
	DIN 4102-1	EN 13501-1	
Minimal non-combustible materials	A1	A1	Gypsum, lime, cement, concrete, minerals, glass, fiberglass, rock wool, ceramic, sand
	A2	A2 – s_1, d_0	Products as in class A1, but containing a small amount of organic material
Small Heavy combustible materials	B1	B – s_3, d_2	Gypsum boards with different (thin) coatings, wood based fire retarders
		C – s_3, d_2	Phenolic foam, gypsum boards with different coatings (less than those in Class B)
Normal Normal combustible materials	B2	D – s_3, d_2	Wood products having a thickness exceeding 10 mm and a density greater than 400 kg/m^3
		E – d_2	Different types of fibreboards, insulating products and plastic products
Large Highly combustible materials	B3	F	Products not tested for fire

All flammable materials in building will release, during fire, a certain amount of heat that will further burden building construction, disable occupants movement towards the exit while simultaneously accelerating fire spread.

This total amount of heat released is called a *fire load*. The fire load can be divided on:

- *immobile fire load* - represents combustible material built in the building construction and the interior of the premises (doors, windows, floor and ceiling coverings, wooden roof structures, etc.), and
- *mobile fire load* - represents an inventory in the premises (furniture, stored goods, equipment, etc.).

Most combustible and conditionally combustible materials in buildings are made of different materials of natural, synthetic and mixed origin. They are an integral part of the building's construction or are located as finishes and furniture in individual premises.

The fire load will be greater as greater the amount of fuel in some space, and to its increase will also contribute materials with high thermal power.

Table 3 shows the heat power of combustible materials for fire load calculation [6]. Fire load is a relative term since constructive element behaviour will not be the same if the heat of burning material, which may endanger the stability of the constructive element, is released in a very short time interval, or this release takes much longer. This will depend on whether burning material is chopped or in pieces, from the humidity of burning material, from the possibility of greater air changes required for proper and complete combustion, the distance between material and heat source and many other factors.

The maximum temperatures in fire are mostly dependent on the fire load in a room. Thus, different fire loads are causing different maximum temperature values. For the description of various fuels combustion in enclosed spaces, for several years there are standard temperature curves that determine temperature dependence on combustion time obtained on the basis of representative fuel tests. Up to date, fire curves have been developed for almost all buildings and situations as well as the fire growth rates.

Table 3
Heat power of combustible materials Hu (MJ/kg)

Typical materials in buildings			
Solids	Hu (MJ/kg)	**Plastics**	Hu (MJ/kg)
Wood	~ 17,5	Polyurethane	~ 23-25
Cellulosic materials (clothes, cork, paper, cardboard, silk, straw)	~ 19-20	Polyurethane foam	26
Wool	~ 23	Polystyrene, polypropylene	~ 40
Linoleum	~ 20	Polyethylene	~ 40-44
Grease	41	Polyester	~ 30-31
Cotton	~ 20	Celluloid	19
Rubber tyre	~ 30-32	Melamine resin	18
Hydrocarbon			
Gases		**Liquids**	
Methane, ethane	~ 50	Gasoline, petroleum, diesel	~ 44-45
Butane, propane	~ 46-50	Oil	41
Acetylene, ethylene, propylene	~ 45-48	Benzene	~ 40
		Benzyl alcohol	33
		Methanol, ethanol, spirits	~ 27-30
Others products			
Solids		**Plastics**	
Bitumen Asphalt	~ 40-41	ABS	~ 35-36
Leather	~ 20	Acrylic	28
Paraffin wax	47	PVC	~ 17-20
Coal, charcoal, anthracite	~ 30	Polycarbonate	29
Rubber isoprene	45	Epoxy	34

Data on the specific fire load density for different occupancies (MJ/m^2 fitting with a Gumbel type I distribution) is presented in Table 4 [12].

Table 4
Fire load density for different occupancies

Occupancy	Mean	80% Fractile *
Dwelling	780	948
Hospital	230	280
Hotel (room)	310	377
Library	1500	1824
Office (standard)	420	511
School	285	347
Shopping centre	600	730
Theatre (cinema)	300	730
Transport (public space)	100	122

*Fractile value of 80% means that 80% of the fire has a lower or equal value of the fire load density than the specified

Table 5 shows the net thermal power and the rate of energy released per square meter of some liquids, plastics and various forms of wood.

Table 5
Net calorific value and heat release rates for different material (Buchanan 1994)

Material	Net calorific value (MJ/kg)	Heat release rate (MW/m^2)
Liquids		
Petrol	43.5	3.27
Light oil	41.9	1.75
Wood		
Flat wood	16.7	0.10
1 m cube	16.7	0.61
Furniture	16.7	6.63
25 mm in crib	16.7	15.3
Plastics		
PMMA	24.9	1.34
Polyethylene	43.8	1.36
Polystyrene	39.9	1.40

Plastics and other synthetic materials generally have a higher calorific value than for example wood, which can cause a higher fire temperature (Buchanan 1994).

Based on the fire load, the amount of available energy can be estimated. On the other hand, reached temperature depends on the speed of the temperature rise during fire. This phenomenon is called the Rate of Heat Release (RHR) which depends on the fire compartment ventilation conditions. Eurocode 1 predicts three speeds of temperature rise: slow (t_{lim} = 25 min), medium (t_{lim} = 20 min) and fast (t_{lim} = 15 min).

Table 6 presents maximum heat release rate [kW/m^2] produced by 1 m^2 of fire in case of fuel controlled conditions depending on the building occupancy.

Table 6
RHR depending on the building occupancy

Maximum rate of heat release RHR_f			
Occupancy	**Fire growth rate**	**t_α (s)**	**RHR_f (kW/m^2)**
Dwelling	medium	300	250
Hospital	medium	300	250
Hotel (room)	medium	300	250
Library	fast	150	500
Office (standard)	medium	300	250
School	medium	300	250
Shopping centre	fast	150	250
Theatre (cinema)	fast	150	500
Transport (public space)	slow	600	250

RHR is the source of gas temperature increase and propulsion power for gas and smoke spread. There are two quite different fire development scenarios depending on the oxygen availability to maintain the combustion process:

- fire controlled by the amount of fuel (RHR achieves maximum value without oxygen limit, so its limited by available fire loads), and
- the fire is controlled by ventilation (the available oxygen is limited by the RHR, if the opening size is too small to allow enough air to enter the fire sector).

Figure 4 illustrates the above scenarios where two completely different RHR curves corresponding to the same amount of fire load as the surfaces beneath both curves are the same.

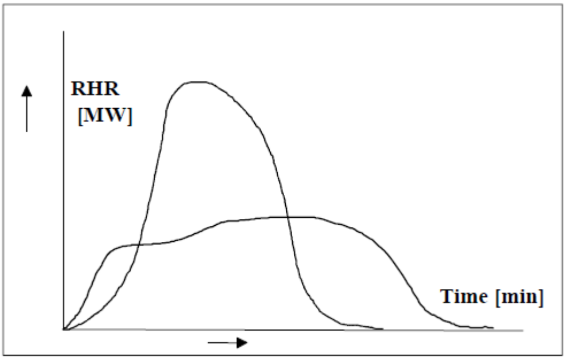

Figure 4. Two RHR curves corresponding to the same amount of fire load [6]

The main cause of destructive fires in buildings is the presence of very large quantities of highly flammable materials, electrical devices malfunction and improperly installed electrical installations (or failures in their maintenance and operation), thermal appliances and other appliances (home appliances without supervision, carelessness in food preparation in kitchens), improperly implemented flue installations, disposed cigarette butt, construction flaws (lack of inspection and control on construction sites, negligence regarding fire protection regulations), intentional fires, etc.

It should be pointed out that in 80% of all fires a human factor directly or indirectly participates as a causative agent because of ignorance, negligence, mistakes in work and improper treatment of the fire occurrence. This data indicates that the focus should be on educating people.

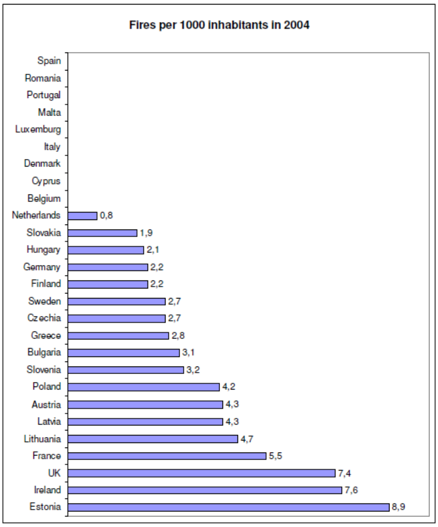

Figure 5. Fires per 1000 inhabitants in EU countries [11]

The most important ignition scenarios understanding is of crucial importance to designers who are trying to reduce probability of fire incidents in buildings. Fire issues are continuously monitored and statistically processed in order to identify trends and characteristics of fires. Due to huge number of possible fire causes, basic classifications and typical cases involving the largest number of fire causes are usually given.

Figure 5 gives an overview of the number of reported fires in 2004 across EU countries (there are no available data for Belgium, Luxembourg, Italy, Spain, Portugal, Cyprus, Malta and Romania). It is noted that the number of fires per 1.000 inhabitants is very different for the 18 EU countries analysed, and that there is a relatively high number in Great Britain, Ireland and Estonia.

Figure 6 gives an overview of the number of deaths and the number of deaths per million inhabitants in EU countries.

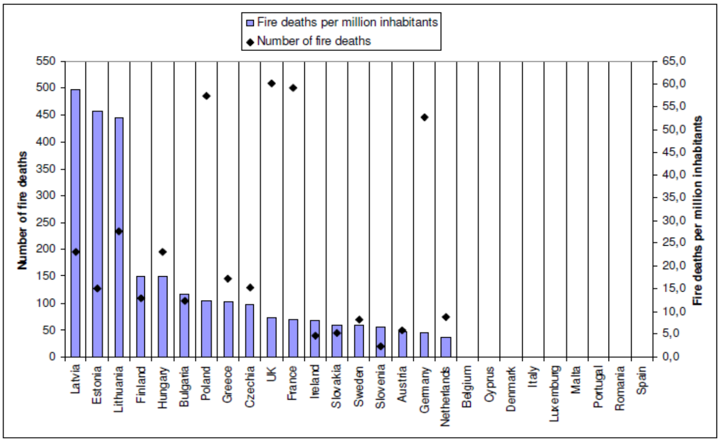

Figure 6. Number of fatalities and fatality rates in EU countries [11]

The (absolute) number of fatalities is relatively high in Poland, the UK, France and Germany. However, compared to the other EU countries the number of fire deaths per million inhabitants is not extremely high in these four countries.

The number of fire deaths per million inhabitants is relatively high in Latvia (58.7), Estonia (54.0) and Lithuania (52.4). The number of fire deaths per million inhabitants is relatively low for the Netherlands (4.3), Germany (5.3) and Austria (5.5). The average fire death ratio is 17.3 for the 18 EU countries studied.

Considering a huge number of possible fire causes, basic classifications and typical cases involving the largest number of causes of fire are usually given. Like all statistical data, data on fire causes also depends on methodology, number of fires, time period, building use etc.

The next classification of fire causes was used to make comparisons between countries more or less possible [11]:

- intentional fire,
- smoking,
- cooking,
- use of candles,
- electric appliance,
- use of a heating appliance,

- imprudence,
- playing with fire,
- other.

Table 7 gives an overview of the cause of fire according to statistics of firefighters in several European and other Western countries, and Table 8 gives an overview of household fires with fatal consequences [11]:

Table 7
General overview of domestic fires

Cause	UK	the Netherlands	US	Australia
Intentional fire	9%	11%	6%	3%
Smoking	57%	3%	2%	4%
Cooking	4%		26%	44%
Use of candles	17%		5%	4%
Electric appliance	6%	31%	3%	12%
Use of a heating appliance			11%	3%
Imprudence				
Playing with fire	4%	2%	0,4%	3%
Other	2%	27%	9%	
Unknown			36%	29%

Table 8
Causes of fatal domestic fires

Cause	UK	London	NL	Sweden	DK	US	Australia	New Zealand
Intentional fire	35,4%		9%	8,3%		11,7%		
Smoking	16,7%	47%	31%	29,8%	51%	7,8%	42%	13,1%
Cooking	5,6%	14%	9%	5,8%		2,2%		16,9%
Use of candles	5,8%	8%	3%	5,0%	9%	5,6%	7%	10,0%
Electric appliance	7,6%	3%	21%	12,4%	4%	3,4%	14%	13,1%
Use of a heating appliance		8%	3%	5,8%		3,4%		6,9%
Imprudence		18%	12%	2,5%				4,6%
Playing with fire	3,3%	3%	6%	0,0%		1,5%	5%	9,2%
Other	23,5%	17%	6%	0,0%		6,8%		13,9%
Unknown		1%	0%	30,6%		57,8%		7,7%

As can be seen from Table 8, the highest number of fatal consequences was caused by smoking (left lit cigarettes/cigarettes butt). Other common causes are recklessness during cooking and the use of defective electrical devices.

According to the Fire Protection Laws, owners and users of buildings and premises are obliged to take care of the correctness of the installation and the equipment that can cause fire and the accuracy and reliability of the smoke alarm, fire extinguishing and fire prevention devices and other protective devices.

1.2.2.Flammability control

The civil engineering development is followed by continuous discovery of new, increasingly decisive solutions in the application of structures and materials that are built in buildings. However, progress in the field of fire safety is not in line with technological innovations and current building practices.

All combustible materials in fire will release a certain amount of heat that will further burden the building structure and accelerate fire spread. There are a lot of cases when the fire was rapidly expanding due to the inadequate finishing materials on the constructions. For this reason, any material used for finishing work on any part of the construction should be such as to limit the flame spread and flammability.

This is mainly controlled by the introduction of a flameproof or flame retardant test, which is performed under conditions defined by some relevant national or international standards. The obtained test results provide a useful indication of a successful ignition control.

Fire endurance test determines the ability of building materials and components to withstand and provide protection during fire.

Numerical modelling of building constructions behaviour due to fire is currently one of the most actual areas of research. The development of simple and efficient numerical models is the basis for a better understanding of the fire as a stochastic process and further improvement of building standards.

Research on the cause of WTC towers collapse in New York in 2001 have shown that passive fire protection systems have been partly destroyed due to airplane strike, i.e. gypsum wall panels were broken up into small pieces and foam fell off the steel members after the attack. The analysis showed that the collapse of the towers was not caused by plane hit, probably because of the numerous strong pillars in the facade, but by the described fire impact on an unprotected construction due to thermal impact.

For this reason the designer should pay particular attention to the proper selection of construction materials and structures in terms of their fire resistance. Since it is impractical to insist on the composition of a structure that will not contribute to combustion in the event of a fire, it is necessary to ensure that these constituent parts of the structure represent as low risk as possible. Also, special surface coatings used for protection against unwanted ignition should prevent heat of the base material as long as possible.

In addition to control the ignition by materials selection in terms of their fire resistance, the designer can use active or passive fire protection measures or both.

1.2.3.Control of fire spread/growth

One of the classic ways to control fire spread is by vertical or horizontal fire compartments. However, they can be sufficient only if there is no possibility of smoke and flames leak at their edges. Also, fire spread may occur in a room or compartment outside the fire origin compartment, if the fire cannot be kept within the limits of its origin because of inadequate closure of the room where the fire occurred [9].

Figure 7 illustrates seven possible ways of horizontal fire spread: above the wall (1), through the openings in the walls (2), through the roof (3), through the roof structure (4), through the lowered ceilings (5), under the double floors (6), through the poorly constructed dividing walls (7).

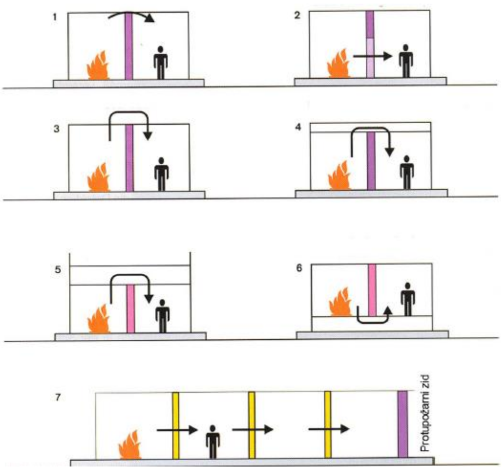

Figure 7. Potential ways of horizontal fire spread

Horizontal fire spreading through the walls can be prevented by fire-resistant walls. In order to prevent fire spread from the room of origin to other rooms through the doors, fire resistant doors must be installed at the boundaries of the fire compartment. Ventilation and other ducts must be made of materials through which the fire cannot be spread, and the prevention of fire spreading is ensured by automatic fire-resistant flaps.

Figure 8 illustrates five ways of possible vertical fire spread: through the floor structures (1), through the openings in the floor structures (2), through the windows (3), through the facade cladding (4), over the burning facade (5).

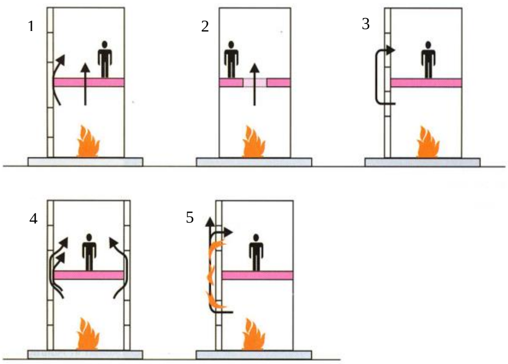

Figure 8. Possible ways of vertical fire spread

In the vertical direction, the fire spreads much faster than in horizontal, and several floors can be affected in 20-30 minutes. This problem occurs in floor structures which are not fire resistant where their can collapse and fall into lower floors. Vertical fire spread through the floor structure is prevented by their fire-resistant construction. There is a danger of a very rapid spread of fire through openings in floor structures, because warm gases that are lighter rise and spread the fire. Such openings should be avoided, and if they cannot be avoided, they should be properly closed.

High risk of vertical spread of fire from one to another fire compartment comes from the facade made of combustible materials. This spreading mode poses a danger to high buildings because the fire is spreading at a higher rate to higher floors and there is a risk of falling pieces of burning materials on the floors below and from the droplet of dissolved materials.

Since the most dangerous time for a building fire occurrence is during renovation, it is necessary to ensure that the used materials create as low hazard as possible.

We will mention a case study of the fire spread that took place in Frankfurt in May 2012 due to inadequate use of thermal insulation materials during the Student Dormitory renovation. Within minutes, until firefighters arrived, the entire building facade was covered with flames, and the air was smoke-black. It took 80 firefighters to put fire under control and ultimately extinguish it. The investigation revealed that the flammable insulation material used to renovate the building quickly expanded fire. Once ignited, this kind of insulating material can create flames and thick smoke and it is likely to drop the hot droplets, which in this case fell on a bunch of inflammable materials that were located below on the construction site (www.firesafeeurope.eu).

Although the fire compartment edges are satisfactory and construction phase is completed, an additional problem may arise in case of the adding installations that can destroy obstacles to fire spread or if the replacement of an obstacle is not in accordance with a satisfactory standard. This situation may also arise when subsequent modifications have been made, due to changes in design, repair or replacement of the materials. When installing installations through walls and ceilings, for which fire protection requirements are set, it is necessary to carry out fire sealing with barriers of the same class of fire resistance as the construction structures through which they pass.

Furthermore, the problem may arise because of the ignition of accumulated flammable waste, which can be ignited in a fire such as Bradford (Anon, 1985, 1986), or may gradually cause flashover with a very slow development of fire (Anon, 1987, 1988). Such problems can be reduced by ensuring the application of fully effective fire protection rules.

1.2.4.Fire protection management

Fire protection management is one of the most important aspects of fire protection in buildings that must be continuously implemented.

In the case of individual use of premises, it is relatively easy to establish procedures with which, in the event of a fire, all personnel would get acquainted with, as well as appropriate persons acting as leaders and directing the fire brigade. However, the problem is in the case of multiple users of space, especially where there is a frequent change of occupancy, where a large number of people pass through, such as, for example, large shopping centres, sports halls, stadiums, etc.

It is therefore essential that owners, usually corporate bodies, establish a fire safety management strategy and ensure the existence of a responsible group of persons who are on duty at all times in order to take full control in the event of a fire outbreak.

This function can be fully performed by adequately trained personnel employed on building safety jobs. It is also necessary to keep a complete record of fire detection, fire control and fire-fighting systems, and to perform complete check of all space to ensure that no action is taken that will put any function of any part of the system out of function. It is important that where fire safety engineering design approach is approved and accepted, the designed measures are applied all the time and no financial pressure will jeopardize fire safety [3].

1.3. Concepts of fire safety

The American National Fire Protection Association (NFPA, 2003) has developed a framework for assessing fire safety, so-called "Fire Safety Concepts Tree". Figure 9 shows a custom overview of the fire safety concept.

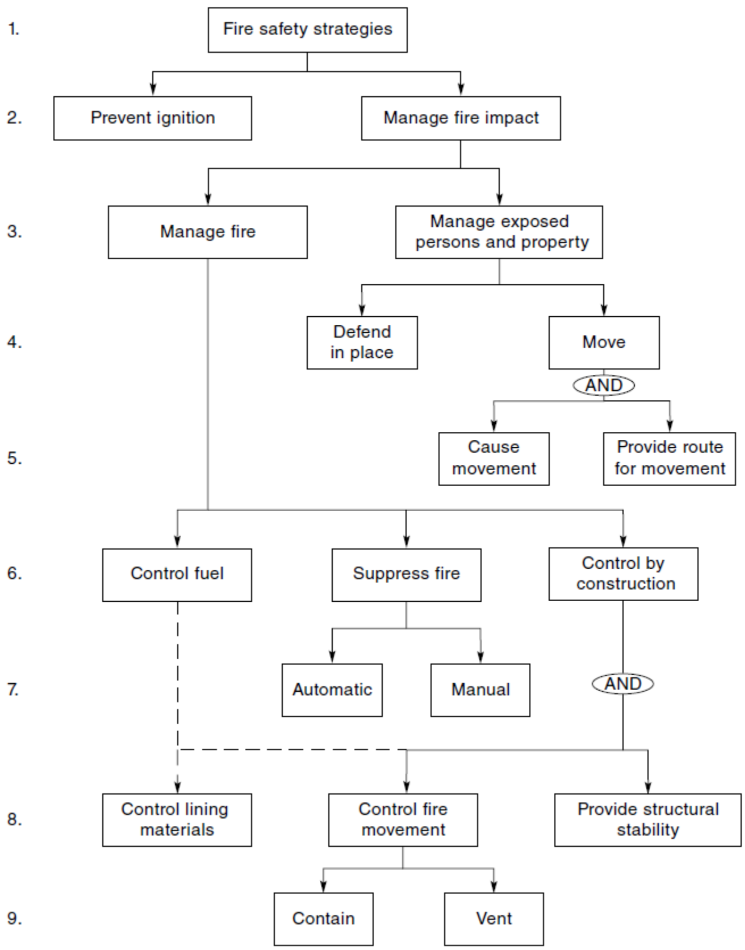

Figure 9. Fire safety concepts tree [2]

Line 2 indicates obvious: there is no need for further fire control if the ignition can be prevented, but if it cannot, it is necessary to control the impact of fire. In reality, it is difficult to eliminate all sources of unwanted ignition, but their probability can be reduced by fire protection measures.

Line 3 indicates that control of fire impact can be accomplished by either controlling the fire or control of exposed people and property. Line 4 indicates that control of exposed persons and property can be accomplished by their evacuation from building or saving them inside the facility. A common strategy is to evacuate people unless they are immobile or otherwise disabled. The overhead solution for high buildings is to move people to safe areas inside the building. Most of the property within the building, which is exposed to fire, must be defended on the spot because it is impossible to execute its rapid relocation.
To begin with evacuation, it is necessary to detect the fire and inform occupants. In addition, there must be adequate and safe ways for their movement (line 5).

Line 6 shows three options for fire control. In first case, the “fuel source” can be controlled by limiting the quantity and geometry of the fuel substance; another option is fire extinguishing; third is fire control with construction measures. Line 7 indicates that fire extinguishing can be automatic or manual, but in both cases the performance depends on early fire detection and use of a sufficient amount of suitable extinguishing agent.

Line 8 of the concept tree shows that fire control by building needs to achieve fire spread control but also ensure stability of the construction. The left field in line 8 points to the fact that the development and size of the fire can be controlled by the limitation of flammable materials in enclosures. It is apparent that it is connected by dashed lines with the “fuel control” field in line 6, as it would be (strictly looking) as its subset, but since the selection and placement of the lining is part of the construction process, it is listed in line 8.
Providing structural stability is important if the building and its sections must be maintained during a fire, depending on the importance of the building. Structural stability is important both for the protection of people and property within a building in case of fire.

Fire spread control can be achieved in two ways by retaining the fire or by taking out the heat (line 9). Removal of heat is a useful strategy to reduce its impact, especially in single-story buildings (or if it is run from the highest floor of high buildings). It can be done by means of an active opening system that is mechanically opened, or a passive system that works on the principle of melting of plastic roof windows.
In both cases, increased ventilation may locally lead to fire growth, but its expansion in the building and total thermal impact on the structure will be reduced. Fire containment to prevent its spread is passive fire protection. Fire resistance helps to limit fire spread from the compartments of origin and at the same time ensures the structural integrity of the fire compartment. Therefore, the walls and floors of most buildings have a certain fire resistance, primarily to keep the fire in the room where it occurs.

Preventing fire development is one of the basic fire protection strategies.

It is also necessary to prevent fire spread on neighbouring buildings by limiting the size of the openings on exterior walls.

Smoke control is achieved by keeping the smoke (closing the space by the barriers) or ventilating the space (natural or mechanical). Ventilation is an important fire strategy when its development is limited by automatic sprinkler systems. Smoke barriers as well as “pressurization” (high pressure ventilation) can be used to control the smoke, as explained below [2].

1.4. Passive fire protection measures

Passive fire protection refers to fire control by system built into structure or fabric of the building. Passive protective measures can be observed through structural protection (constructive elements heat protection), compartmentalization and protection provided by the building envelope (surrounding walls and roof). These elements last a buildings lifetime and are always available as a fire protection.

Passive fire protection of buildings does not require any external power and it relies on construction features, use of materials, products and building elements that should meet specified fire performance requirements.

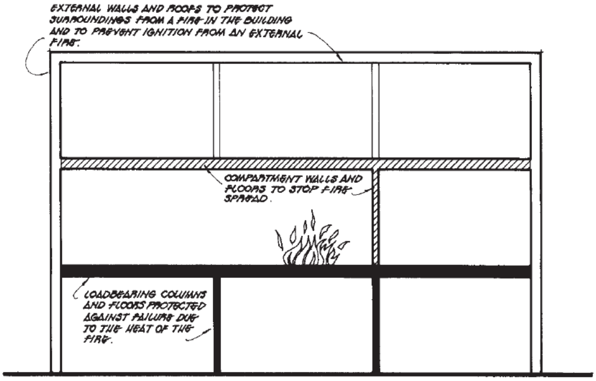

Figure 10. Passive fire protection [4]

1.4.1. Structural protection

The main cause of fatal casualties in fires is smoke suffocation (the number of smoke casualties is from 80% to 90%[1]), while a significantly less people die due to building collapse. This relatively low incidence of deaths due to “structural collapse” does not mean that structural integrity (load bearing response) is irrelevant. On the contrary, all buildings must be designed in such a way as to maintain their structural integrity during the fire and thus enable safe evacuation of people and provide a certain level of protection for firefighters.

Time period before the collapse of the building should be minimal [8]:

- 15 minutes: for lightweight wooden or steel structures, including roofs,
- one hour: for low buildings (up to 3 floors) masonry / concrete construction, and
- three hours: for high buildings (more than three floors) masonry/concrete construction.

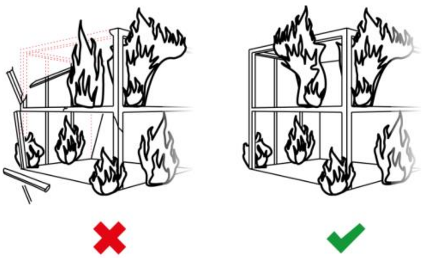

Figure 11. Structural integrity in fire [8]

[1] M. Vidaković, B. Vidaković, „Požar i arhitektonski inženjering“, Beograd 2008.

The specified nominal period may vary depending on the risks arising from the building use, the level of fire risk, fire load and the durability of the materials used.

Required fire protection level for constructive elements depends on the established need for evacuation and fire extinguishing. If the building has to “survive” until all occupants are evacuated then the required time is short, about half an hour; however, if the safety strategy of a people’s life is based on sheltering in the “safe zones” - area within the building or it is necessary for firefighters to intervene in it, then protection requirements will increase for 1 hour or more. If insurance companies prefer to renovate the building, than rebuild and reconstruct it, then the fire protection requirement grows for 2 or 4 hours.

The ability of a constructive element to maintain its function when exposed to heat is its *fire resistance* expressed in the unit of time. This includes fire resistance of assemblies, not just elements, which must be calculated.

The fire resistance that must be ensured depends on fire load in the building.

Table 9 can be used as an example for estimate of fire load and required fire resistance for certain types of buildings. Its purpose is mainly informative and educational because it gives a rough estimate.

It is important to bear in mind that the structural protection is as good is the weakest link in the project and that the connections between the constructive elements must have equal fire resistance as the elements themselves. Special risks are associated with high buildings over 10 floors or buildings with underground premises in two or more levels.

Table 9
Fire load and fire resistance for various building types

	Building type	Fire load	Required fire resistance (min)		
			1-floor	2-flors	3-or more floors
1.	Houses	Low	0	30	30
2.	Flats	Medium	30	30	60
3.	Residential institution (hospital, prison, etc.)	High	30	60	90
4.	Hotels, boarding-house	Medium	30	30	60
5.	Office, school	Medium	30	30	60
6.	Assembly and recreation (cinema, theatre)	High	30	60	90
7.	Shops	Medium	30	30	60

Customized tables 5.1 and 5.2 [4]

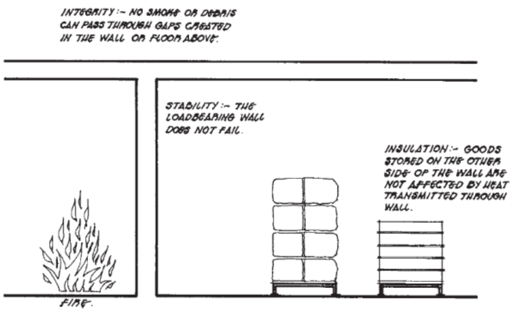

Figure 12. Stability, integrity and insulation [4]

Fire resistance is measured by the ability of an element or structure to fulfil required load capacity, integrity and insulating properties in conditions of predicted fire for a certain period of time. It is these three requirements that define fire resistance: *the loadbearing capacity* of the structure is its dimensional stability, its *integrity* is its ability to resist thermal shock and cracking, and to maintain its adhesion and cohesion; *insulation* is property of the material defined by the level of heat conductivity.

For structural elements in first moments of fire, only stability and integrity are important, but since they often have the role of dividing the building into horizontal floors or vertical walls to prevent the fire spread, then the insulation characteristic is also significant.

1.4.2. Compartmentalization

From the point of view of securing people’s lives and protecting the property, it is necessary to adequately divide the building into fire compartments which prevent the fire and smoke spread for a certain period of time. Each large building should be divided into compartments vertically, horizontally or combined.

Structures and elements separating fire compartments are:

- fire walls (internal and external), which are constructed as continuous structures from floor to roof, and
- other separating structures and fire and smoke resistant elements at the compartment boundary.

Fire walls are one of the most effective fire protection solutions, as they prevent fire and smoke spread within an object (or fire compartment), between buildings, but also buildings and flammable materials that may be near or placed on the façade.

As a barrier to heat and smoke spreading through openings that cannot otherwise be closed, *smoke curtains* are being used. There are also cases when this type of protection is practical from aesthetic or justified for technical reasons. Simply put, smoke curtain is a specially constructed curtain that goes down from the ceiling to block the openings and stop fire and smoke spread between two areas. Smoke curtain is largely similar to a metal shutter by vertically lowering it from the “upper box”, however, because it is made of materials such as glass fibres, it is much more flexible and compact. Auto-curtain-based solutions (connected to a fire fighting system, automatically descend in fire) are available, fixed (permanently in place and used to divide space into fire compartments) or insulating (providing additional isolation, allowing people to go closer to curtains and they do not feel the impact of fire heat on the other side).

Figure 13. Fire curtains for elevators
(http://www.metalpress.co.il/en/product/fire-curtains-for-elevators.aspx)

Fire compartment, by definition, is an enclosed space that is separated from adjacent spaces inside the building by structural elements that have prescribed fire resistance, preventing fire and smoke to expand within the building or adjacent buildings within a certain time. A fire compartment can be one room, a group of rooms or even an entire building. The design of the fire compartments depends on purpose of the building, height and fire load as well as on fire resistance of the building. The fire compartment is the basic spatial unit, which is independently treated in fire risk assessment, during preparation of fire protection plans, and operations during firefighting.

The fire compartments boundaries must be of non-combustible building materials. In order to achieve a complete separation of the fire compartments, it is necessary to protect elements that are not constructive, such as the interior walls and doors. At the boundaries of the fire compartment, there must be no "weak points" or cavities that would compromise the smoke and fire barriers; all installations that pass through the walls or flooring of the compartment must be designed in such a way as to provide the same level of fire resistance as all segments of that compartment.

Except that the built-in doors must meet the defined fire resistance of fire compartment, it is necessary to take measures to ensure their quick closing in case of a fire.

The great danger, throughout the lifetime of building, is possibility of compromising the fire compartment by unconsciously cutting its boundaries or by adding openings.

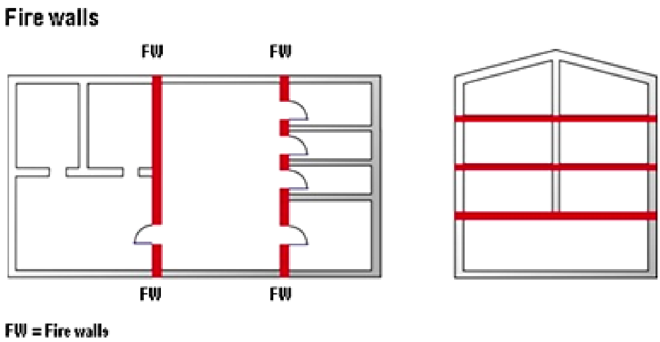

Figure 14. Building division into compartments
(http://www.fml.eu.com/compartmentation/)

Normative and construction regulations mainly address issues of fire compartment size, but the designer must be aware of some basic principles on which an adequate division of compartments is based on.

Number of fire compartments in a building depends on the fire load, installed systems for automatic alarm and fire extinguishing, the occupancy of space and the purpose of the building because many regulations indicate the maximal area or volume depending on the purpose of the building.

Below is a table, which is of educational and informative character, with the rough values of fire compartment size for different types of buildings.

Table 10
Areas of fire compartments for different types of buildings

	Building type	Size of compartment
1.	Houses	Each flat separate
2.	Flats	Each flat separate
3.	Residential inst. (hospital, prison, etc.)	1600 m^2
4.	Hotels, boarding-houses	2500 m^2
5.	Offices and commercial	2500 m^2
6.	Assembly and recreation (theatre, cinema)	1600 m^2
7.	Shops	2500 m^2

Customized table [4]

When content in the building is more inflammable, the size of the fire compartments should be smaller. When dividing the building into compartments the compartment geometry is not important, it is imperative to preserve the integrity of.

If each floor has to be a separate fire compartment, then it must be ensured that the exit from each floor on the staircase has the same fire and smoke resistance. For housing units, each unit must be a separate fire compartment.

Fire compartment boundaries can also be conditioned by the ability of the occupants to escape from endangered areas. The maximum acceptable length of the evacuation path may also limit the fire compartment size. Apart from dividing the building into compartments based on fire loads, the protection of evacuation paths from the building which must be considered and they must be treated as separate fire compartments.

1.4.3.Fire spread between structures

One of the passive fire protection roles is to limit the risk of fire impact to neighbouring buildings and people outside the building and reduce the risk of fire in adjacent facility. Preventing fire spread on adjacent buildings refers to: nearby buildings and buildings that touch exterior walls.

The safe distance represents a free space around a building that is wide enough to prevent the fire spread to adjacent buildings.

In case it is not possible to provide the necessary distance to the boundary of the parcel, or already built buildings, then the solution are firewalls.

Firewalls do not only serve to separate two buildings that are closer to each other than it is safe but can also divide a larger building into “fire sectors” and thus prevent fire spread as explained in previous subchapter.

To limit the fire spread, special attention should be paid to roof and exterior walls (due to thermal radiation). The *roof* poses a risk of convective currents that can carry burning particles (wooden materials, etc.) to the adjacent buildings. There are standards for designing resistant roof structures even when exposed to direct flame or heat radiation.

Exterior walls also require special attention in design phase because the possibility of transferring heat through them to adjacent buildings (Figure 15b). It is common to limit the risk of heat radiation by reducing the number and size of the openings in exterior walls when they are close to other buildings. Limiting the size of the opening in order to reduce radiant heat to neighbouring buildings can also help in preventing the spread of fire among floors of the same building.

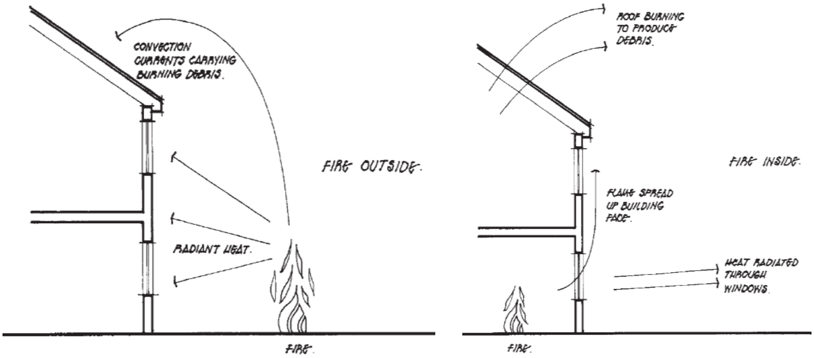

Figure 15. Envelope protection (a-fire outside, b-fire inside) [4]

Risk of fire spread on the *building surface* is mainly minimized by selecting facing materials and roof coverings with zero fire propagation (brick, concrete and stone). Some building regulations limit the use of facade materials with high fire propagation potential.
In its manual UNOPS [8] gives recommendations to prevent fire spread among buildings (Figure 16).

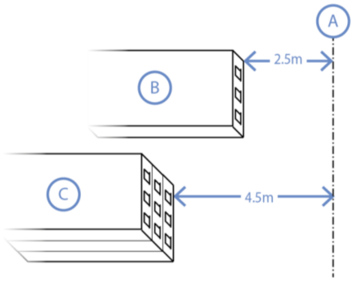

Figure 16. Fire separation from adjoining property
(A – adjoining property boundary, B – single story building, C – three story building)

With the aim of controlling fire spread, the distance between the new building and side or rear boundary of the property must be at least 2.5 m for single story buildings and 1.0 m for each additional floor if the wall of the new building contains windows facing the boundary.

If such fire retention is not feasible, the wall must not have windows facing the boundary of property and must be of a non-combustible construction.
If the exterior wall has an opening with windows at an angle of 90° to the boundary, then it must have a separation wall (fire-resistant wall) of at least 600 mm (Figure 17-B).

There is no need for fireproof properties of an external wall or window on it when it is located at the minimum required (or greater) distance from the boundary wall (Figure 17-C).

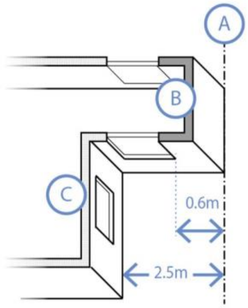

Figure 17. Fire separation and windows: boundary (A), 2-hrs fire wall (B), not fire rated wall (C)

1.5. Active fire protection measures

Active fire protection refers to fire control by a person or an automatic device. Implementation of active measures implies installation and maintenance of fire detection and alarm equipment, smoke and fire control systems; installation of firefighting systems; dangerous contents control and a fire protection control system installation (central).

In order to ensure people lives during evacuation, it is necessary to provide means for fire detection and control. Fire control is necessary to minimize smoke production, which allows more efficient evacuation, as well as to limit the temperature rise in the structure / building and thus reduce subsequent damage. A large number of buildings are equipped with automatic fire extinguishing systems due to the requirements of insurance companies or as a compromise between active and passive systems envisaged by state regulations.

1.5.1. Fire detection

Fire detectors and alarm systems are the basic fire protection elements of each building whose installation and use can significantly reduce the loss of human lives and property in fire.

The types of detectors most commonly used in buildings, especially when human lives are exposed, are heat and smoke detectors, and detectors of other fire phenomena, as well as combined (multisensory) detectors.

Fire detection systems can be manual or automatic or their combination.

Figure 18. Fire detection and alarm systems (http://www.vindexsystems.com)

Manual systems are relatively simple: a traditional glass panel whose breaking triggers an alarm. These systems require the presence of people, who need to detect a fire and assess its seriousness, and can be installed and used only where it is certain that people will be present.

In areas where people sleep, it is necessary to install additional detection devices - automatic. Such buildings are hotels, hostels, accommodation facilities with special care, as well as multi-purpose facilities.

Automatic systems are activated due to the presence of large amounts of heat or smoke which are registered by sensors and directly include a fire extinguisher (such as a sprinkler) or indirectly activate any fire control or evacuation system. Newer versions of these systems are made using low power lasers or infrared sensors to monitor smoke occurrence. Many automatic systems are based on a combination of heat and smoke detection sensors. The advantage of automatic devices is that they can alert people who are in the building even before them itself notice fire.

All detection devices, other than those installed in buildings with a small number of occupants, should be connected to a system that:

- indicates the place of fire or the location where the alarm was triggered,
- activates fire control by closing the fire resistant door, smoke curtain or automatic ventilation system, and
- initiates evacuation procedures with the automatic registration of a fire occurrence in a local fire department.

1.5.2.Smoke control: ventilation and pressurization

Smoke that is produced in fire should not obstruct visibility during evacuation, and the lower limit to which it descends should not be below 2.5-3 m from the floor area during first 15 minutes of fire (Building Research Establishment, 1987; Morgan and Gardner, 1991).

Smoke control is necessary because of problems caused by the toxic substances present in it as well as due to the complete disorientation effect due to reduced visibility. Figure 19 shows fire development without smoke extraction.

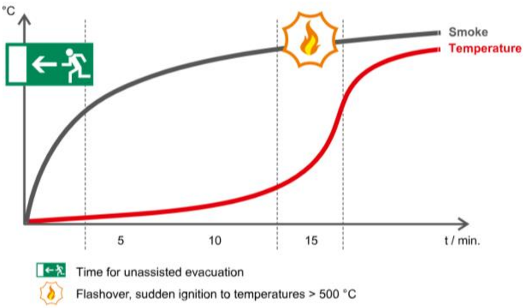

Figure 19. Typical fire description without smoke extraction (www.hautau.de)

The simplest way to prevent smoke spread inside the building is to extract it out of the building, retaining the smoke in area where it is produced, but also providing time for evacuation and fire extinguishing measures.

The fact that the designer must especially take into account is smoke layering (stratification) caused by the buoyancy, which is disturbed by its cooling. As shown in Figure 20, an upper layer of hot smoky gases (zone A) is formed just below the ceiling and is floating on the smoke-free air below (zone B). The cloud of smoke rising from the fire pulls the air as it rises and creates the upper layer (Zone C).

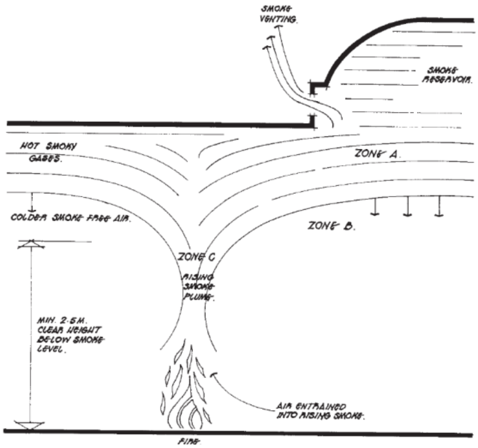

Figure 20. Smoke stratification and ventilation [4]

With the fire growth, the amount of smoke will increase exponentially, and the smoke layer will become thicker as the fire develops. Ventilation system has the role of ensuring that the amount of smoke that is added to already formed layer is well balanced with the one that is vented, so that the depth of smoke layer remains constant and never drops to a level that would endanger occupants encountered in the fire.

The restriction of horizontal smoke spread can be achieved by smoke curtain-barriers that descend from the ceiling. These curtains can be permanently mounted or simply activated in the event of a fire (Figure 21).

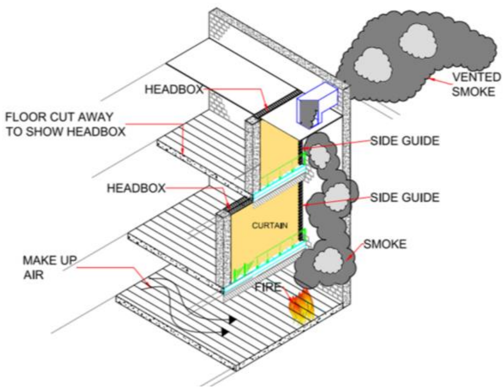

Figure 21. Smoke venting and smoke curtains (http://www.fercoshutters.com/smoke-curtains/)

The simplest way to design venting for single-story buildings is through roof openings – *natural ventilation*, while in high-rise buildings *mechanical* systems - *forced ventilation* should be installed.

Natural ventilation fully responds to environmental conditions, and outdoor weather conditions determine the efficiency of this ventilation system. Mechanical ventilation must be designed with a high level of reliability, with conditions to be repairable and maintained throughout the life of the building. Forced ventilation is functioning by pulling smoke from a room with fire or evacuation corridors, by blowing fresh air into the room or as combination of mechanical extraction and blowing.

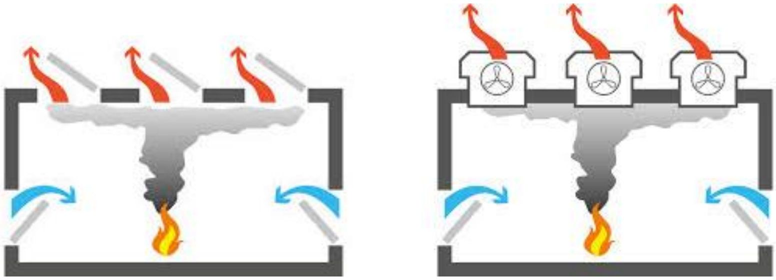

Figure 22. Natural and forced smoke extraction (www.hautau.de)

Previously it is emphasized that the problem of door installation in fireproof walls can be solved by adequate door selection of appropriate fire resistance. Nevertheless, such doors must be opened during escape and evacuation, where there is a risk of smoke entering the protected area.

This type of risk can be prevented by a stairway with protected lobby/corridor approach, which creates an “air chamber” (Figure 23) where at any one time only one door will be opened.

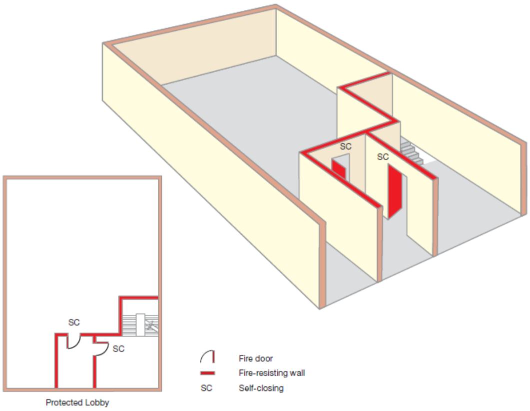

Figure 23. Examples of a stairway with protected lobby/corridor approach [5]

A better method for protection than this is by *increasing pressure (pressurization)* in such protected zones, corridors or stairs. One of the possibilities is smoke extraction from areas intended for evacuation, but this solution is far from ideal because of danger for drawing large quantities of smoke into this area.

Pressurization is suitable for spaces of less volume, not only to prevent the penetration of smoke during the fire, but also to maintain “clean atmosphere” free of contaminants such as in operating rooms or electronic equipment factories (Figure 24).

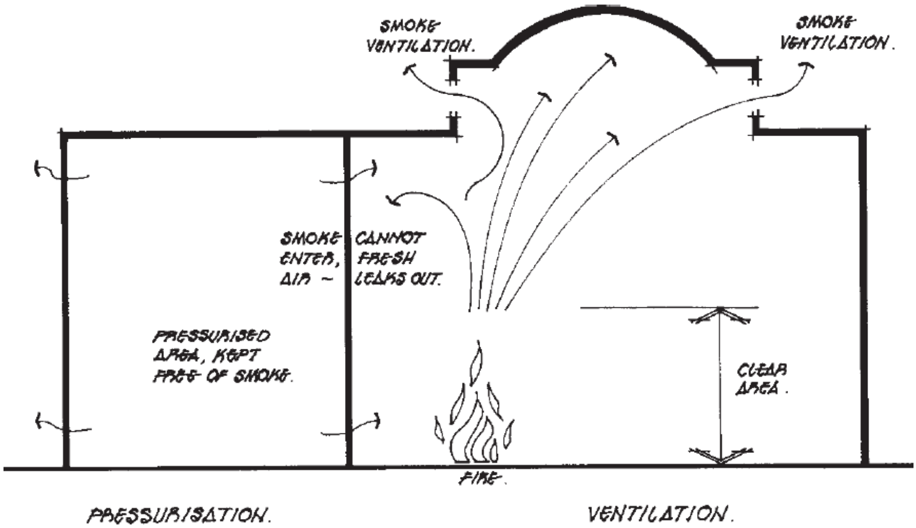

Figure 24. Pressurization and ventilation [4]

Fresh air is supplied to the room of the pressurization and the pressure is maintained at a level higher than in the surrounding rooms. When the door in the area with positive pressure opens the air will come out of this room outside rather than smoke flowing in. Additionally, when the door is closed the positive pressure that has been achieved will prevent leakage through this area and fresh air will leak out into adjacent areas.

It is possible to pressurize only stairs, but it is obviously much better (and the leakage will be significantly reduced) if the staircase lobbies are under pressure. The ideal solution is to place the entire evacuation route under higher pressure, including horizontal and vertical sections.

1.5.3.Fire extinguishing

Three are main ways of using fire extinguishers: by persons who encounter a fire with manual fire extinguishing equipment; automatic fire extinguishing systems; and finally, extinguishing by fire department. Architects and designers must consider and anticipate which of these ways can be expected during the design phase, and then design to ensure their efficiency.

The architect has an important role in providing adequate manual fire extinguishing equipment (handheld appliances, fire blankets and hoses) in sufficient number and at the required locations.

The number and arrangement of *manual fire extinguishers* depends on fire risk assessment.

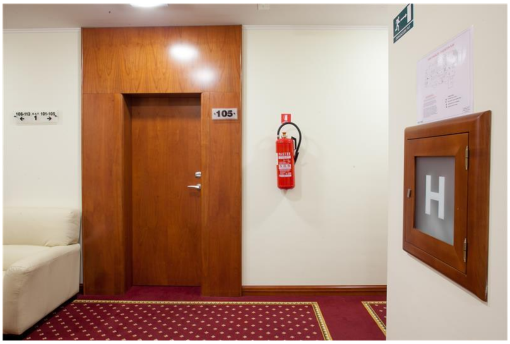

Figure 25. Fire extinguisher and hydrant in the hotel room (http://www.travelino.hr)

Fire extinguishers should be installed in places where they are easily accessible in a fire. They are located near the entrances to each compartment, they are on evacuation routes and available to persons entering that area to extinguish the fire.

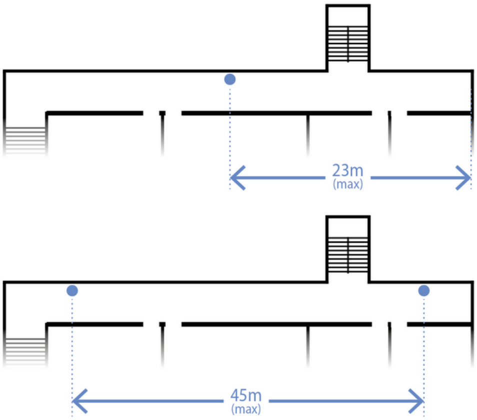

Figure 26. Travel distance between fire extinguishers [8]

According to the recommendations of UNOPS, the maximum space per unit is 500 m^2, and the maximum length of the route for movement between the fire extinguishers is 45 m or 23 m if the building contains only one extinguisher [8].

Fire extinguishing in buildings is also carried out by a special water supply network - a *hydrant network* with special connectors used when the initial fire extinguishers cannot extinguish the fire and when larger quantities of extinguishing agents are required.

There are three types of hydrants: wall hydrants, above-ground and underground; and the network is designed as an internal and external hydrant network. They are installed in accessible places in buildings, mainly in stairways, in cabinets in such a way that the entire protected area can be covered by a jet of water.

The distance of external hydrants is determined depending on the purpose, size and characteristics of the building.

Figure 27. Internal and external (above ground) hydrants

For all buildings with an area exceeding 500 m^2 are required fire hose reels and standpipes for firefighting. They must include at least 30 m long fire hose 25 mm in diameter located on the reel and a separate 38mm outlet capable that can supply 60 l/min of water for at least 30 minutes. The hose must have a range of 6 m from the farthest part of the building (furthest from the hydrant location). The hydrant location must be at the exit or exit stairway from the building. If the distance meets these requirements, one hydrant can be used to serve a two-story building. Determination of the number of lines, hose reels and water supply lines is determined by the expected use of the facility, the number of occupants and other installed fire protection systems [8].

In the initial stage of the design of the building, firefighting professionals should be consulted in order to confirm that all fire protection measures are fulfilled, the requirements for fire vehicles access roads (Figure 28); location of external hydrants, their number and capacity; as well as the provision of communication links for alarm activation notification.

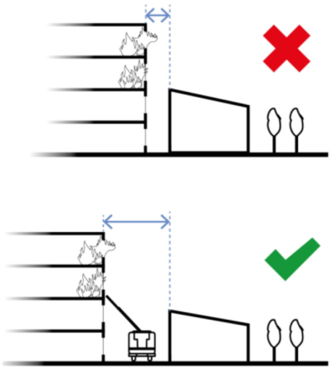

Figure 28. Firefighting vehicle access [8]

Automatic fire extinguishing systems are systems that are triggered by fires without any human activity. The most common form of automatic extinguishing is sprinkler installation, and some other forms for specific risks are also available.

Sprinkler installations have been in use since the late 19th century and are the most widespread type of fixed fire extinguishers. They are used for automatic fire extinguishing in different types of buildings that may be of different fire loads (hotels, museums, schools, shopping malls, etc.). They are considered to be very effective in reducing property losses to such an extent that insurance companies provide significant discounts to building owners where sprinklers are installed (up to 60-80%).

Sprinkler installations can be a wet system used in premises where there is no danger of freezing or water vaporization and where the pipeline is always filled with water, and a dry system in which piping is filled with air under pressure. They can be mounted on the ceiling or on the walls; sometimes they have to be set up even at different heights (e.g. warehouses where the goods are kept in piles at different heights).

Sprinkler systems are designed for extinguishing small fires or for controlling fire until the fire service arrives. The system is activated by cracking a glass ampoule located in the heat-sensitive sprayer head, releasing the water through the area under the sprinkler head after it is activated. More sprinkler heads will be activated if the temperature increases locally.

The sprinkler system will prevent small fires from escalating into large, and can also extinguish fires.

Figure 29. Sprinkler head (https://www.houseidea.pw)

The contribution of sprinklers to human life safety is difficult to quantify; their value lies in limiting the fire and enabling occupants to evacuate.

In the tests carried out after the Woolworth[2] Fire (Anon, 1980; Stirland, 1981), the maximum floor-level temperatures with installed sprinkler sprayers were 190° C and without sprinkler installation 940° C; the volume of smoke and gases produced in first 7 minutes with sprayers was 1500 m^3 and without them 10.000-20.000 m^3. It is also estimated that sprinklers would provide extra minutes for evacuation and that the fire would be brought under full control in 22 minutes. There were also some dilemmas regarding the efficiency of the sprinkler system in fire because there were also cases when they did not activate, for which evidence is not completely clear. Stirland (1981) points out that such suspicion are unnecessary.

Besides sprinkler fire control device, *drencher (deluge)* systems are being used and can be activated manually or connected to a fire detection system. Drencher systems are mostly installed on the outside of the building (although they may be internal) to protect it from the transfer of fire from adjacent buildings. They are usually placed on roofs or through windows or other openings.

Drencher systems are used in areas with high fire load, or where very rapid fire spread is expected. They are very similar to sprinkler systems because they have water jets with pipelines, valve stations, and a water supply system. They differ from the sprinkler because the drencher nozzles are open and do not have a thermally sensitive element, so when system is activated the water runs through all the nozzles and not only locally as with the sprinkler.

[2] Anon (1980) Report of the Woolworth's fire, Manchester. Fire Prevention, 138, 13–24

Figure 30. Drencher system (www.themeparkpro.co)

Drenchers can also be used to cool the structural elements (e.g. steel roof beams), and are especially valuable as additional protection on the openings within the compartment walls. These systems are mainly used when very fast fire spreading is expected. It also serves to separate the compartment affected by fire (water curtain); also finds application in low flash point fluid reservoirs cooling as well as for fire extinguishing in these plants. Drencher installations can also be used to apply a fire extinguishing foam.

Water curtains are a special example of a drencher installation use whose task is to prevent fire spread from one to another compartment (Figure 31). Water curtains are formed by the water stream from the drencher installation and their activation is carried out through a fire alarm control unit. Water curtains provide additional safety when dividing space into the fire compartments.

Figure 31. Water curtain (www.coopersfire.com)

In addition to the above-mentioned automatic systems, there are stable systems for extinguishing fires with inert gases, water fog, foam, carbon-dioxide, special fire protection systems for kitchens, etc. also in use. Some systems, due to the toxicity of the extinguishing agent and the reduction of oxygen content in the space, are not applicable in buildings where people reside, while systems that use foam and powder in some cases cannot be used due to the impact of the extinguishing agent on the building content.

1.5.4. Interaction between active and passive protection

There are evident negative consequences of interaction between ventilation devices and sprinkler systems. The problems identified by Heselden (1984) are caused by effect of the water cooling the smoke, which prevents its upward buoyance. A number of tests

(Hinkley and Illingworth, 1990; Hinkley et al., 1992) have been conducted in connection with this phenomenon and design guidelines have been published (Morgan, 1993).

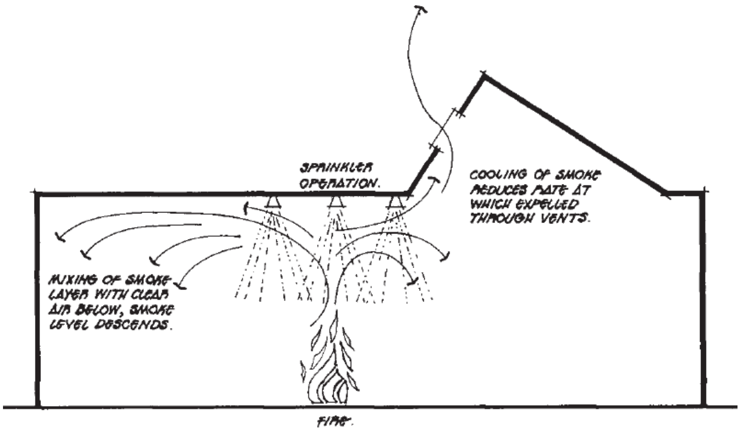

Figure 32. Sprinkler and ventilation interaction [4]

The problem caused by the interaction of the venting system and sprinkler installation is manifested in a way that the cloud of smoke does not rise and therefore reduces visibility during evacuation; or in the other case, the flow upward due to ventilation causes loss of water drop effects that fall from the head of the sprinkler device. Day (1994) indicates that, where both systems are installed in storage, sprinklers should be activated before the smoke extraction device, and in other areas where evacuation is important, both systems can act together.

In addition to possible negative influence of sprinklers on the ventilation systems operation, it is possible and vice versa. If the ventilation system is activated before sprinklers activation, their action may be delayed. However, in most cases, this delay in activation is considered negligible.

1.6. Evacuation

In the seventies of the last century extensive research was conducted in the United States and England concerning people behaviour in fire; followed by research in Canada and Japan, and then in Germany in the 1980s. The research, which included fires in residential buildings, hospitals, business buildings, hotels, theatres, shopping malls, gave different results that were in direct correlation with the tradition related to the construction of buildings (wooden houses or brick buildings), building maintenance, awareness or people discipline in relation to fire safety.

Evacuation is one of the most important issues in fire, and each building must be designed so that occupants can escape on their own or with the help of rescuers/other persons. All occupants must be able to come to a safe place before being overcome by smoke or fire, or the time needed for escape must be shorter than the fire spread time, which can be achieved by fire and smoke spreading control, and providing escape routes that are not too long or too complex.

There are two basic fire escape strategies: first, *egress* - simple, straightforward escape from the building when an alert is announced; and secondly, *refuge* - the use of structural fire protection to secure a safe place within the building so that evacuation is carried out from the compartment where a fire occurred to the adjacent sector. There is also a third strategy: rescue by people from outside the building (applies only to low-rise buildings, not applicable to chronic patients and infirm, as well as disabled people).

Figure 33. Evacuation stairs [1]

Under the term “means of escape” are meant structural means to provide safe ways for moving from any part of the building to a safe place. Means of escape are, therefore, the most fundamental of the above-mentioned fire protection measures required to secure life, and must be planned at an early stage of building design, since there are many factors affecting them (Figure 34).

When designing, it is important to consider future potential occupants and their behaviour patterns, which may have a greater impact than some of the physical design factors that are defined in codes and guidelines.

The basic characteristics of building occupants are their number, mobility, space awareness, fire alarm response, and sleep risk (buildings where people are sleeping are more dangerous than those used only during the day).

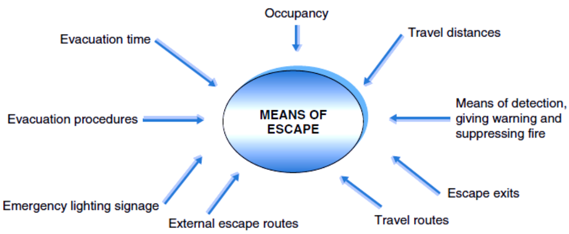

Figure 34. Factors affecting means of escape [1]

“Safe place” is mostly located outside the building; the escape on flat roofs or similar structures from which there is no way to the street level is not an escape path in the modern sense of the term (although previously accepted under certain circumstances). Nor windows are means of escape, except in some situations when they are accepted in building regulations as alternative means of escape from some rooms in single-story or two-story residential buildings.

Evacuation of disabled persons should not rely solely on rescue by firefighters and rescue services; in this case, for buildings with two or more floors, primary means of escape should be suitable evacuation lifts with provided the alternative routes are via stairs.

1.6.1.Requirements for evacuation and escape

All buildings should meet the basic requirements for evacuation and escape in case of fire: adequate evacuation route, clearly marked paths, sufficient number of exits, proper door opening, sufficient width of exit doors, sufficient corridor and staircase width, properly designed blind corridors, necessarily illumination.

Below are the illustrations and definitions as stated by the UNOPS in the Design Planning Manual of 2014 [8].

- Travel distance: clearly marked door to exit from the building or staircase from upper floors, maximum travel distance up to 40 m from the furthest point on the floor.

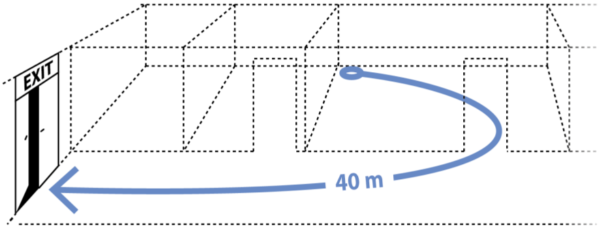

Figure 35. Maximum travel distance

- Signage visibility: signs for exit should be noticeable and visible; must also be foreseen in the corridors where the exit doors are obscured from view.

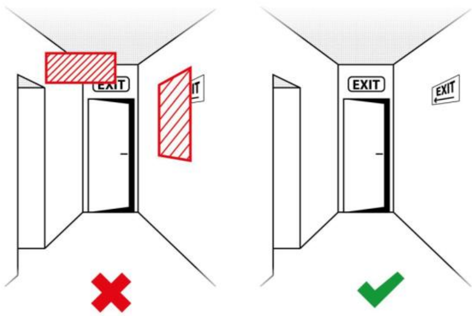

Figure 36. Signage visibility

- Number of exits: Minimum per floor should be in two locations, widely separated to provide alternative escape points. Buildings with more than 500 people per floor require 3 exits, and over 1.000 people per floor require 4 exits. The exceptions are small buildings with less than 100 m^2 and 20 occupants per floor, and for a maximum two-story building can have one exit.

- Exit opening direction: In all rooms where more than 20 occupants can be accommodated, the exit door must be provided and must be opened in the direction of the exit path (Figure 37). Other doors can be opened inwards; if only one door is provided, they must be opened in the direction of the exit.

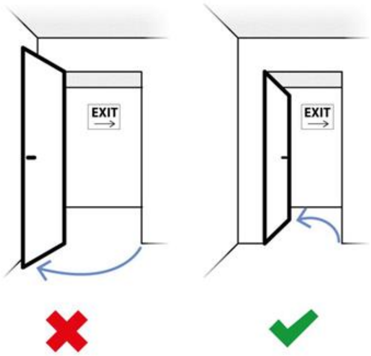

Figure 37. Exits opening direction

- Exit door width: the width of all exit doors (or multiple doors such as double swing doors) must be appropriate for expected number of people. The minimum clear width of the exit door is 900 mm inside the frame (Figure 38). This applies to all exit doors in exit path from the from the room egress door to the exit of the building, ensuring that all doors are accessible to wheelchairs with the prevention of emergency door locking.

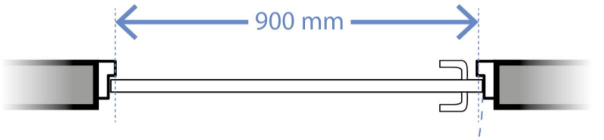

Figure 38. Exit door width

- Corridor and stairway width: all the width of the stairway and the corridors must be appropriate for the expected number of people. The minimum light width of the corridor is 1.500 mm, and the staircase is 1.200 mm between walls faces (Figure 39). Opening the door in light width is not allowed. This also applies to external access to the balcony when they are used primarily for egress.

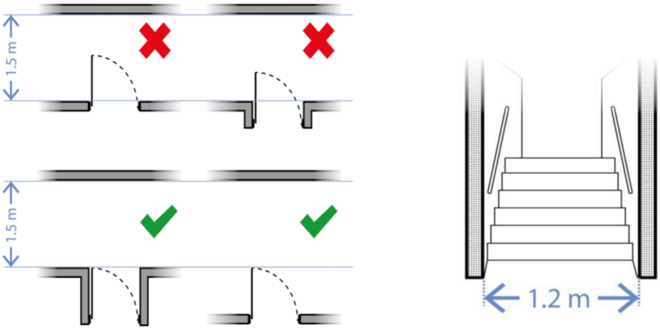

Figure 39. Minimum corridor and staircase width

The only exception to the stairs width requirement is that stairs serving less than 50 occupants can be 900 mm wide between the walls.

- Dead end corridors must not be longer than 6 m where they branch from the main egress corridor (Figure 40). The exception to this rule is in conditions when only one exit is needed.

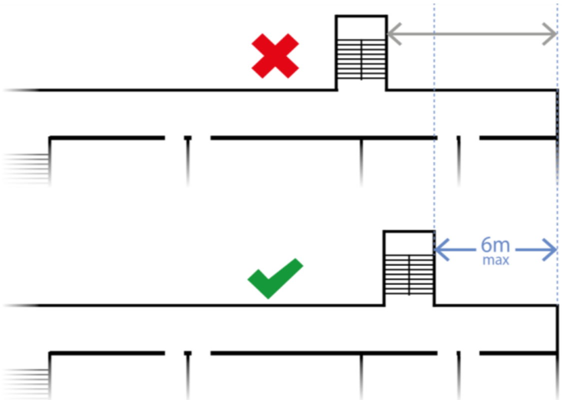

Figure 40. Dead end corridors

- Fire isolated stairs: if the building has more than 3 floors, one of the exit stairs must be insulated from fire. This is done with suitable construction techniques to achieve at least 2 hours of fire resistance including all doors, windows, wall and floor construction. The fire isolated stair exit at the ground floor must lead directly to the outside of the building or through a corridor with fire resistance of 2 hours.
- Safety lighting: all buildings MUST be equipped with a backup light with a spare battery to ensure safe evacuation in case of power failure (Figure 41).

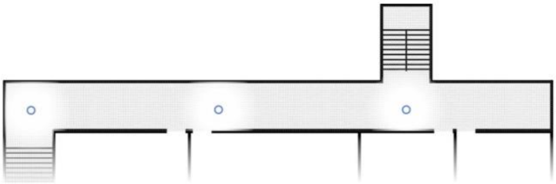

Figure 41. Emergency lighting

It is necessary to determine the assembly points for evacuated people who must be far enough from the building to avoid injuries due to falling debris from glass or walls (Figure 42).

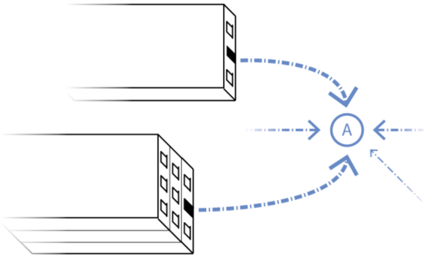

Figure 42. Assembly points

Special attention should be paid to guarded facilities and shelters in circumstances in which it is not certain or desirable that those persons leave the buildings (prisons, etc.).

When considering the adequacy of exit capacities, it is necessary to understand the term “separate exits” and the 45º rule. Exits can be considered separate only if one is in relation to the other, viewed from any point in the room concerned, at an angle of at least 45º. Alternative escape routes are available from point C because the angle ACB is 45º (or more), and the CA or CB path (whichever is shorter) should not be greater than the maximum length of the evacuation path defined for alternate routes. Alternative route is not available from D because angle ADB is less than 45º. There is no alternative way from point E (Figure 43).

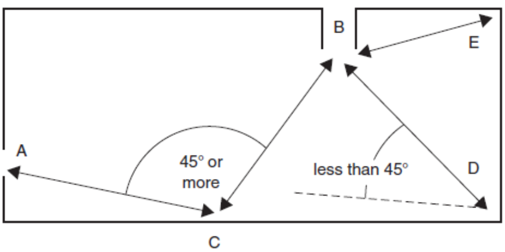

Figure 43. The 45º rule [1]

Alternative evacuation routes should be at an angle of at least 45º (as explained above) or, if this is not the case, they must be separated from each other by a fire resisting construction.

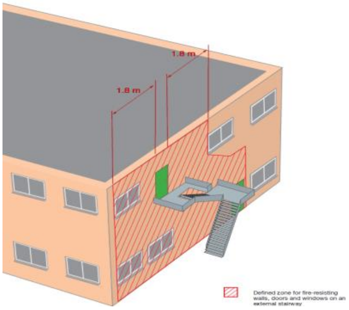

Figure 44. External stairway [5]

In order for the external stairway to be considered as an escape route, it must be protected from the impact of fire from the building all its entire length [5], which means that all doors, windows and walls must be fire resistant (Figure 44). Windows should be fixedly closed and the doors self-closing. This stairway should not be used under normal circumstances.

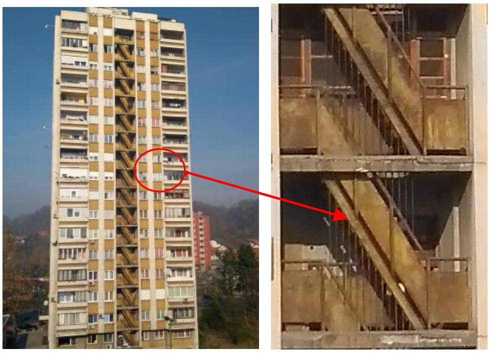
Figure 45. Consequences of poor maintenance of an external fire escape stairway (high-rise building Tuzla, B&H)

External fire escape stairway should be protected against external influences (moisture, ice, etc.) and made of non-slip materials (Figure 45).

It is sometimes justified to use the roof for the escape route. In this case, additional precautions are needed: the roof must be flat, the movement path must be clearly defined and well illuminated and protected (guarded with barrier); the evacuation path over the roof must be constructed of fireproof materials; the exit from the roof must lead to a safer place where people can be fully rescued.

Figure 46 shows an illustration of a typical evacuation route over the roof of the building.

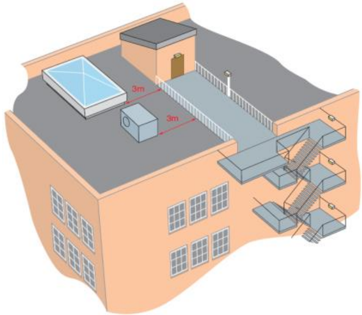

Figure 46. An escape route across a roof [5]

1.6.2. A prescriptive and engineering approach to evacuation

Human life rescue is the most important goal of all fire safety measures, and for its realization, adequate evacuation plans are needed.

Two approaches are used in evacuation planning [7]:

- prescriptive method (based on regulations), and
- performance method.

The first method (a) is based on three main points:

- density of people,
- their flow, and
- evacuation routes length and width.

The second method (b) determines whether required escape time is less than available safety egress time.

- *Prescriptive approach*

Most of building codes and fire protection standards currently used are prescriptive. They draw their roots from the XIX century when large fires imposed the need for specific building construction.

These codes were made without an effective assessment of their adequacy, excessiveness or collision with other requirements. Thus, regulations arisen based on empiricism and experience, rather than on scientific understanding of fire. In the meantime, a lot of progress has been made in fire protection, but all the knowledge is not embedded in everyday practice.

The prescribed methods for evacuation safety assess are based on the following:

- number of exits and maximum width and length of the evacuation paths,
- maximum time for evacuation, and
- a management strategy for maintaining rescue paths accessible and safe.

It is assumed that the occupants walking speed is about 0.5 m/s and the exit time is about 3-5 min. These values are satisfactory for most situations, but in some cases they may be insufficient, and then an engineering approach is necessary.

- *Engineering approach*

The performance method is based on the definition and comparison between the available time to arrive at a safe place, ASET (*available safe escape time*) and the time required for occupants to reach the safe place of the RSET (*required safe escape time*). Engineering approach implies determining the safety boundary, which is the difference between ASET and RSET time (Figure 47):

$$T_{safety} = T_{ASET} - T_{RSET} \qquad (1)$$

The performance method is applicable in complex or modern buildings where the prescriptive approach is not acceptable. In addition, the engineering approach can be used to estimate and evaluate solutions resulting from prescriptive methods.

In order to calculate ASET time it is necessary to study the fire in detail from ignition to its development. This is the time between fire outbreaks and time when "tenability criteria" have been exceeded due to smoke, toxic substances and heat. The end of the ASET time is the time when the conditions in space are considered untenable because occupants can no longer save themselves.

RSET time depends on 4 different "times" that are in direct relation with occupants and their physical and behavioural characteristics.

Those times are:

- detection time - t_{det} (time from the moment of fire ignition to its detection by either manual or automatic system),
- alarm time - t_a (time from detection to alarm activation),
- pre-movement time - t_{pre} (time from detection to the moment when first occupant starts moving),
- walking time - t_{tra} (time occupants take to move from where they are to a safer place).

$$t_{RSET} = \Delta t_{\det} + \Delta t_a + \left(\Delta t_{pre} + \Delta t_{trav}\right) \qquad (2)$$

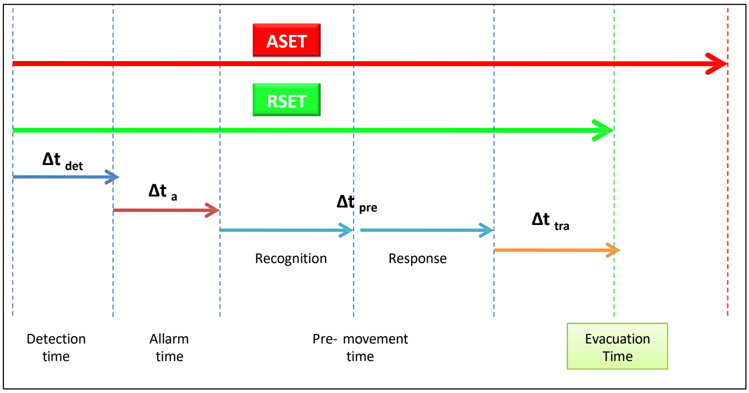

Figure 47. Evacuation time [7]

Four times are strongly influenced by human behaviour, which is why it is not easy to identify them. The analysis takes into account occupant behaviour in real and simulated emergency situations.

For safe escape, it is crucial to precisely design the escape routes (in relation to the distance to the safe place) and the escape time.

Choosing the most appropriate approach and method of calculation for an appropriate evacuation design is responsibility of the engineers with various available options: from the simplest manual calculations to the most sophisticated software simulations, depending on the objective and desired accuracy level.

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PROPERTIES OF BUILDING MATERIALS AT ELEVATED TEMPERATURES

Vesna Bulatović, Mirjana Malešev

1. INTRODUCTION

The temperature increases during the fire and nonuniform temperature field forms. For that reason the structural material deteriorates at different levels in the interior of the structure [1]. They undergo physicochemical changes, accompanied by transformations in their microstructure and changes in their properties [2]. When the structure acts as the loadbearing and support system of the building, it should maintain sufficient strength for a certain time period during a fire, so that firefighters can fight the fire, rescue the injured and deceased victims safely, and save valuable property [1].

Several types of materials may be used in structural engineering such as timber, steel, reinforced concrete, brick etc.

2. MATERIALS PROPERTIES AT ELEVATED TEMPERATURES

Selecting the appropriate materials and assemblies to meet expected fire conditions in a structure requires familiarity with fire properties of building materials and structural assemblies [3]. The behaviour of a structural member exposed to fire is partially dependent on the thermal and mechanical properties of the material of which the member is composed. While calculation techniques for predicting the process of deterioration of building components in fire have developed rapidly in recent years, research related to supplying input information into these calculations has not kept pace [2].

Generally, building materials may be burnable (combustible) and nonburnable (noncombustible). Noncombustible materials will degrade under the higher temperatures of a fire but will not burn. Combustible materials will not only degrade at higher temperatures but also ignite and burn, thereby adding to the fuel contents during a fire [4]. For combustibility is very important following: flame spread, fuel contributed and smoke development [3].

Most building materials are not stable in the temperature range 20°C to 800°C, and collecting information on the properties of building materials throughout this temperature range is not easy. The thermophysical and mechanical properties of most materials change substantially within this temperature range associated with building fires. For example concrete at 500°C is completely different from the material at room temperature [2], steel are very sensitive to heat and lose half of their strength at 500°C and size of wooden structures is reduced as a result of combustion and forming carbon layer [5].

Research on the application of materials in this field has numerous difficulties. Most of the properties are temperature dependent and sensitive to testing method parameters such as heating rate, strain rate, temperature gradient, and so on. Harmathy [2] cited the lack of adequate knowledge of the behaviour of building materials at elevated temperatures as the most upsetting trend in fire safety engineering. It is important to use values that ensure agreement between experimental and analytical results and know how to utilize and extend information based on previous considerations and gathered from the technical literature.

Knowledge of unique material characteristics at elevated temperatures is critical to determine the fire performance of a structural member. These properties are discussed in the following sections.

Approximate melting temperature for some materials are given in Table 1 [6].

Table 1
Approximate melting temperature of materials [6]

Materials	Approximate melting temperature of materials (°C)
Polyethylene	110-120
Lead	330
Zinc	420
Aluminium alloys	500-650
Aluminium	650
Glass	600-750
Silver	950
Brass and bronze	850-1000
Copper	1100
Cast iron	1150-1300
Steel	>1400

2.1. Mechanical properties

The basic mechanical properties that determine the fire performance of structural members are strength, modulus of elasticity and creep of the component materials at elevated temperatures.

Stress-strain relationships

The mechanical properties of solids are usually derived from standard tensile or compressive tests. The strength properties are usually expressed in stress-strain relations, which are often used as input data in mathematical models for calculating the fire resistance. The variation of stress with increasing strain (deformation) is shown in Figure 1, for a metallic material. Because of a decrease in the strength and increase in the ductility of the material, the slope of the stress-strain curves decreases with increasing temperature [2].

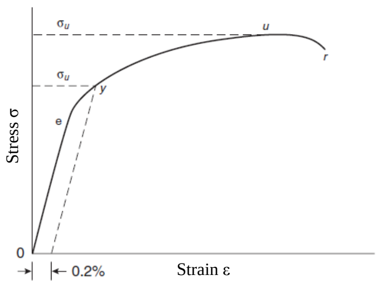

Figure 1. Stress-strain curve for metallic material [2]

Section 0-e of the curve in Figure 1 represents the elastic deformation of the material, which is instantaneous and reversible. The slope of that section is the modulus of elasticity, E (Pa). Between points e and u the deformation is plastic, nonrecoverable. The plastic behaviour of the material is characterized by the yield strength at 0.2 percent offset, σy (Pa), and the ultimate strength, σu (Pa). The tensile or compressive strength of the material is generally expressed by means of yield strength and ultimate strength. After some reduction of cross-sectional area, the test specimen ruptures at point r [2].

Figure 2 shows the variation of strength with temperature, precisely ratio of strength at elevated temperature to that at room temperature, for concrete, steel, wood and FRP (Fibre Reinforced Polymer). For all materials, the strength decreases with increasing temperature; however, the rate of strength loss is different. For materials such as concrete, compressive strength is of main interest since it has very limited tensile strength at higher temperatures. However, for materials such as steel, both compressive and tensile strengths are of equal interest [2].

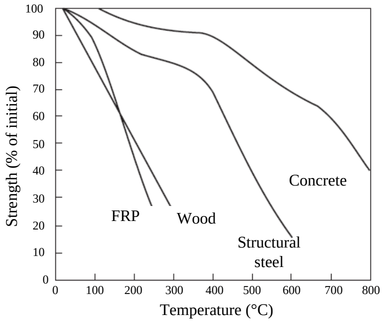

Figure 2. Variation of strength with temperature for different materials [2]

Modulus of elasticity

The modulus of elasticity (E) expressed as the ratio of the deforming stress to the strain in the material, and is a measure of the ability of the material to resist deformation. Generally, the modulus of elasticity of a material decreases gradually with increasing temperature.

Creep

Creep is defined as the time-dependent plastic deformation of the material and is denoted by εt. It is the term which describes long term deformation of materials under constant load, but at normal stresses and ambient temperatures, the deformation due to creep is not significant. Contrary, at higher stress levels and at elevated temperatures, the rate of deformation caused by creep can be substantial. For most materials, creep becomes noticeable only if the temperature is higher than about one-third of the melting temperature. The main factors that influence creep are the temperatures, the stress level and their duration. The complementary term is "relaxation" which describes the reduction of stress in materials subjected to constant deformation over a long period of time [6].

If the load is removed after a while there will be recovery of some of the creep deformations, as shown in Figure 3 (a). Creep becomes more important at elevated temperatures (Figure 3 (b)) because creep can accelerate as load capacity reduces, leading to secondary and tertiary creep [6].

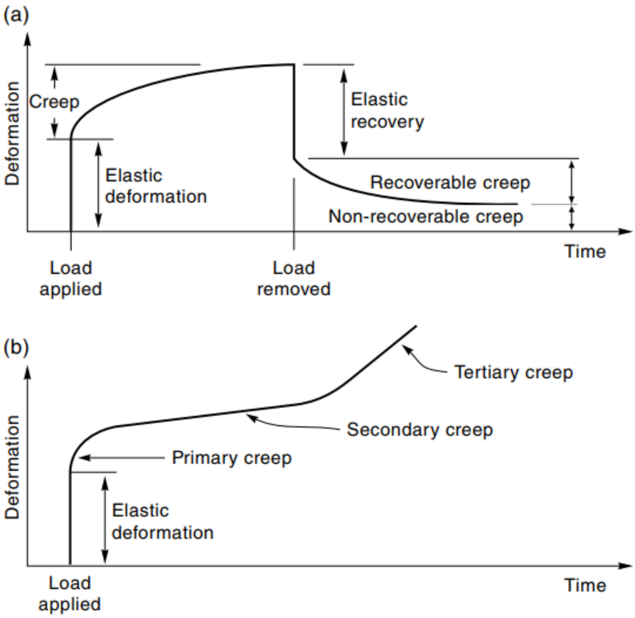

Figure 3. Creep in structural materials: a) creep under normal conditions, b) creep at elevated temperatures [6]

2.2. Thermal properties

The main material properties that influence the temperature rise and its distribution in a member are:

- thermal conductivity,
- thermal expansion,
- specific heat,
- thermal diffusivity
- mass loss, and
- density and porosity.

These properties depend on the composition and characteristics of the constituent materials.

Thermal conductivity

The coefficient of heat conduction of a material is defined as the quantity of heat (J) passing through per unit area (m^2) with uniform temperature within a unit of time (h) and per unit temperature gradient (K/m). Its units are W/(m·K) or W/(m·°C). The temperature rise in a member, as a result of heat flow, is a function of the thermal conductivity of the material. In porous solids, like most building materials, the mechanism of heat transmission is a combination of conduction, radiation, and convection. The thermal conductivity will depend not only on the conductivity of the solid matrix, but also on the porosity of the solid and the size and shape of the pores. If pore size is less than about 5 mm, the contribution of pores to convective heat transmission is negligible. At elevated temperatures, because of the increasing importance of radiant heat transmission through the pores, conductivity becomes sensitive to the temperature gradient [1].

Experimental data indicate that porosity is not a greatly complicating factor as long as it is not larger than about 10%. However, with insulating materials, the porosity may be 80% or higher. If the solid is not oven-dry, a temperature gradient will induce migration of moisture, mainly by an evaporation condensation mechanism. The migration of moisture is usually, but not necessarily, in the direction of heat flow, and manifests itself as an increase in the apparent thermal conductivity of the solid. Even oven-dry solids may undergo decomposition reactions at elevated temperatures. The heat created in this way adds to the complexity of the heat flow process as they move in the pores [1].

The thermal conductivity of layered, multiphase solid mixtures depends in which direction the phases lie in relation to the direction of heat flow and is determined using the simple mixture rule. At higher temperatures, because of radiative heat transfer through the pores, the contribution of the pores to the thermal conductivity of the solid must not be disregarded. The thermal conductivity of solids is a structure sensitive property. For crystalline solids, the thermal conductivity is relatively high at room temperature, and gradually decreases as the temperature rises. For predominantly amorphous solids, on the other hand, the conductivity is low at room temperature and increases slightly with the rise of temperature. The conductivity of porous crystalline materials may also increase at very high temperatures because of the radiant conductivity of the pores [1].

Since the heating of the material causes the nonreversible microstructural changes, the thermal conductivity of that material is usually different in the heating and cooling cycles.

Thermal expansion

The thermal expansion characterizes the expansion (or shrinkage) of a material caused by heating. It is defined as the expansion or shrinkage of unit length of a material when it is raised one degree in temperature. The expansion is considered to be positive when the material elongates and is considered negative when it shortens. The thermal expansion of a material is dependent on the temperature.

Thermal strain is the expansion that occurs when most materials are heated. Thermal strain is not usually important for fire design of simply supported members, but must be considered for complex structural systems especially where members are restrained by other parts of the structure and the thermal strains can induce large internal forces [6].

Specific heat

The specific heat of a substance defines the amount of heat it absorbs as its temperature increases [7]. The specific heat or heat capacity (Cp) of a body is defined as the amount of heat required to raise the temperature of unit mass by one degree Celsius [1]. The units are J/kg·K. The concept of specific heat is normally associated with solids and liquids, but it is equally applicable to gases. Specific heats are required for calculating flame temperatures. Values for a number of important gases at constant pressure and a range of temperatures are given in Table 2 [1].

Table 2
Heat capacities of selected gases at constant pressure [1]

Temperature (K)	Cp (J/mol·K)				
	298	500	1000	1500	2000
Species					
CO	29.14	29.79	33.18	35.22	36.25
CO_2	37.129	44.626	54.308	58.379	60.350
H2O(g)	33.577	35.208	41.217	46.999	51.103
N_2	29.125	29.577	32.698	34.852	35.987
O_2	29.372	31.091	34.878	36.560	37.777
He	20.786	20.786	20.786	20.786	20.786
CH_4	35.639	46.342	71.797	86.559	94.399

If experimental information is not available, the Cp versus T relationship can be calculated from data on heat capacity and heat of formation for all the components of the material tabulated in a number of handbooks.

Thermal diffusivity

The thermal diffusivity of a material is defined as the ratio of thermal conductivity to the volumetric specific heat of the material. It measures the rate of heat transfer from an exposed surface of a material to the inside. Similar to thermal conductivity and specific heat, thermal diffusivity varies with temperature rise in the material. For the larger diffusivity, the faster temperature rise at a certain depth in the material. Thermal properties of some materials are given in Table 3 [6].

Table 3
Thermal properties of some materials [6]

Temperature (K)	Thermal conductivity λ (W/mK)	Specific heat Cp (J/kgK)	Density ρ (kg/m^3)	Thermal diffusivity α (m^2/s)
Copper	387	380	8940	1.14×10^{-4}
Steel	45.8	460	7850	1.26×10^{-5}
Brick	0.69	840	1600	5.2×10^{-7}
Concrete	0.8-1.4	880	1900-2300	5.7×10^{-7}
Glass	0.76	840	2700	3.3×10^{-7}
Gypsum plaster	0.48	840	1440	4.1×10^{-7}
PMMA	0.19	1420	1190	1.1×10^{-7}
Oak	0.17	2380	800	8.9×10^{-8}
Fiber insulating board	0.041	2090	229	8.6×10^{-8}
Polyurethane foam	0.034	1400	20	1.2×10^{-6}
Air	0.026	1040	1.1	2.2×10^{-5}

Mass loss

The mass loss is often used to express the loss of mass at elevated temperatures. Thermogravimetric curve is a plot of change in mass relative to the temperature. A thermogravimetric curve consider reactions accompanied by loss or gain of mass but, it does not consider changes in the microstructure or crystalline order of materials.

Density

The density ρ (kg/m^3) is the mass of a unit volume of the material, comprising the solid itself and the air-filled pores, in an oven-dry condition.

Thermal conductivity increases most often as the density increase and very often density determines how quickly flame spread across the surface of material. So, the flame spread rate across the surface of a heavy material is usually slower than that across a light material [5].

Thermal inertia

One of the important properties of material which connects conductivity, density and heat capacity is called thermal inertia and given as their product. So, for example, brick and concrete have high thermal inertia which allows more heat to be accumulated into them thereby lowering the hot gas temperature. In contrast, insulating materials have a low thermal inertia [8]. In this material with a low thermal inertia the surface heats up quickly as less heat is conducted in the material and ignites after a considerably shorter time than material with high value of thermal inertia. It is opposite for material with high thermal inertia [5].

2.2. Special properties

Beside thermal and mechanical properties, certain other properties, such as spalling in concrete and charring in wood, influence the performance of a material at elevated temperature. These properties are unique to specific materials and are crucial for predicting the fire performance of a structural member and overall systems.

Critical temperature

The critical temperature is defined as the temperature at which the material loses much of its strength and can no longer support the applied load. For example, North American standards (ASTM E119) assume a critical temperature of 538°C for structural steel and this temperature is also regarded as the failure temperature in the calculation of fire resistance of steel members. The concept of critical temperature is used for reinforced and prestressed steel in concrete structural members for evaluating the fire resistance ratings. These ratings are generally obtained through the provision of minimum member dimensions and minimum thickness of concrete cover. The minimum concrete cover thickness requirements are intended to ensure that the temperature in the reinforcement does not reach its critical temperature for the required duration. For reinforcing steel, the critical temperature is 593°C, while for prestressing steel the critical temperature is 426°C [2].

Spalling

Spalling is defined as the breaking of layers (or pieces) of concrete from the surface of the concrete elements when the concrete elements are exposed to high and rapidly rising temperatures, such as those experienced in fires [2]. Spalling may depend on several mechanisms or combinations thereof such as pore pressure, stresses due to temperature gradients, differences of thermal dilatation and chemical degradations at elevated temperatures [9].

Figure 4. Spalling in concrete [1]

It may occur soon after exposure to heat (usually within 30 min of severe fire exposure [9]) and can be accompanied by explosions, or it may happen when concrete after heating has become so weak that pieces fall off the surface. The consequences may be limited as long as the extent of the damage is small, but extensive spalling may lead to early loss of stability and integrity. This phenomenon is very complex and cannot be predicted with simple mathematical temperature models [9].

Highstrength concrete (HSC, concrete with compressive strength in the range 50 to 100 MPa) [2] is considered to be more susceptible to spalling than normal-strength concrete (NSC, concrete with compressive strength in the range 20 MPa to 50 MPa). Probable reason for that is its low permeability and low water/cement ratio. In a number of test observations on HSC specimens, it has been found that spalling is often of an explosive nature and should be properly accounted for evaluating fire performance. Spalling in NSC and HSC columns is compared in Figure 5 using data obtained from full scale fire tests on loaded columns [2].

It can be seen from Figure 5 that the spalling is quite significant in the HSC. Spalling is probably caused by the build-up of pore pressure during heating. The extremely high water vapour pressure, generated during exposure to fire, cannot escape due to the high density (and low permeability) of HSC. This pressure build-up often reaches the saturation vapour pressure.

The pressure reaches approximately 8 MPa at 300°C, and such internal pressures are often too high to be resisted by the HSC mix having a tensile strength of approximately 5 MPa. The drained conditions at the heated surface with the low permeability of concrete, lead to strong pressure gradients close to the surface. When the vapour pressure exceeds the tensile strength of concrete, parts of concrete fall off from the structural member. Spalling is depending on the fire and concrete characteristics. However, other researchers explain the occurrence of spalling on the basis of fracture mechanics and argue that the spalling results from restrained thermal dilatation close to the heated surface.

(a) (b)

Figure 5. Spalling in a) NSC and b) HSC after exposure to fire [2]

Spalling, which often results in the rapid loss of concrete during a fire, exposes deeper layers of concrete to fire temperatures, thereby increasing the rate of transmission of heat to the inner layers of the member, including the reinforcement. When the reinforcement is directly exposed to fire, the temperatures in the reinforcement rise at a very high rate, leading to a faster decrease in strength of the structural member. The loss of strength in the reinforcement together to the loss of concrete due to spalling, significantly decreases the fire resistance of a structural member.

In addition to strength and porosity of concrete mix, density, load intensity, fire intensity, aggregate type, and relative humidity are the primary parameters that influence spalling in HSC. The variation of porosity with temperature is an important property needed for predicting spalling performance of HSC [2].

Charring

One important distinction is between materials that are char forming and those that are not. Charring is the process of formation of a layer of char at the exposed surface of wood members during exposure to fire [2]. The char layer that forms on the surface of materials as they burn acts to reduce the rate of heat transfer to the interior of the material where pyrolysis is occurring and consequently reduces the rate of pyrolysis and burning over time.

This reduction in heat transfer is due to both the insulating characteristics of the char layer as well as reradiation from the char surface which can heat up to temperatures well above the pyrolysis temperature of the material. For materials that do not form a char layer, the surface temperature will be at or near the pyrolysis temperature. Materials that do not char tend to burn at higher rates than those with forming the char layer. This is due to the higher net heat flux at the pyrolysis front for materials that do not char [10].

Wood goes through thermal degradation during heating, which includes the conversion of wood to char (layer without strength) and gas resulting in reduction of the density of the wood. Analyses have shown that the charring temperature for wood lies in the range of 280°C to 300°C. This process may occur in FRP and plastic members [2].

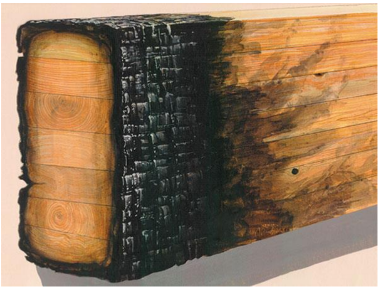
Figure 6. Charring of wood [2]

The charring rate is defined as the rate at which wood is converted to char. It is a critical parameter in determining the fire resistance of a structural wood member. In the standard fire resistance test, it has been noted that the average rate of charring transverse to the grain is approximately 0.6 mm/min but the charring rate parallel to the grain of wood is approximately twice the rate when it is transverse to the grain. The most important parameters that affected on charring are density, moisture content, and contraction of wood. The charring rate decreases with increasing density of the wood and also with increasing moisture content. The charring rate in real fires depends on the severity of fire to which the wood is exposed. Generally, the fire resistance of the member depends on the extent of charring and the remaining strength of the uncharred portion [2]. The amount and depth of charring is commonly used by investigators to evaluate fire spread, intensity, and duration of the fire [11].

3. CONCRETE

Concrete members and systems generally behave well in fires. According to some tests it is possible to achieve very high levels of fire resistance, up to 4 hours [12]. Cover concrete which is poorly cracked or has spalled off, may be replaced with poured or sprayed concrete with incorporated additional reinforcing if necessary. Sometimes concrete members without visible damage may have reduced strength due to elevated temperatures of the concrete or the reinforcing. The thermal behaviour of concrete depends on many factors, some of them are: the component materials, mineral chemical composition, mix proportion and moisture.

Concrete has the low thermal conductivity and because of that the heat affected region is often not very thick. Loss of strength of the concrete itself is usually of less concern than loss of strength of the steel reinforcing. In simply supported flexural members, the compression zone on the top of the slab or beam is often not exposed to very high temperatures. Loss of strength of concrete near the surface can be estimated with an impact rebound hammer.

Most types of concrete change colour after heating, depending on the aggregate. Marchant (1972) describes a design procedure for reconstruction of fire damaged reinforced concrete buildings and reports following facts: for typical concrete during heating:

- concrete heated up to 300°C will have no colour change,
- concrete heated to 300–600°C may be pink,
- concrete heated to 600–950°C may be whitish-grey, and
- concrete heated over 950°C may be a buff colour.

Concrete suffers no significant loss of residual strength when heated below 300°C, whereas for higher temperatures the strength loss will depend on the temperature in concrete. After cooling, the concrete slowly returns only part of its strength (Lie, 1992) [6].

Concrete is excellent and proven fire resistance properties deliver protection of life, property and the environment in the case of fire. Whether it is used for residential buildings, industrial warehouses or tunnels, concrete can be designed and specified to remain robust in even the most extreme fire situations [13].

Using concrete in buildings and structures offers exceptional levels of protection and safety in fire [13]:

- Concrete does not burn, and does not add to the fire load;
- Concrete has high resistance to fire, and stops fire spreading;
- Concrete is an effective fire shield, providing safe means of escape for occupants and protection for firefighters;
- Concrete does not produce any smoke or toxic gases, so helps reduce the risk to occupants;
- Concrete does not drip molten particles, which can spread the fire;
- Concrete restricts a fire, and so reduces the risk of environmental pollution;
- Concrete provides built-in fire protection – there is normally no need for additional measures;
- Concrete can resist extreme fire conditions, making it ideal for storage premises with a high fire load;
- Concrete's robustness in fire facilitates firefighting and reduces and delays the risk of structural collapse;
- Concrete is easy to repair after a fire, and so helps businesses recover sooner;
- Concrete pavements stand up to the extreme fire conditions encountered in tunnels;
- Concrete elements have excellent thermal inertia and massivity, and thanks to them concrete may withstand high temperatures for a very long time, with a minimum of deformation [14].

Table 4
Thermal properties of some materials [6]

Unprotected construction material	Fire resistance	Combustible	Contribution to fire load	Rate of temperature rise across a section	Built-in fire protection	Reparability after fire	Protection for evacuees and fire-fighters
Timber	Low	High	High	Very low	Very low	Nil (Zero)	Low
Steel	Very low	Nil (Zero)	Nil (Zero)	Very high	Low	Low	Low
Concrete	High	Nil (Zero)	Nil (Zero)	Low	High	High	High

Concrete structures have a reputation for good behaviour in fires. As it is known concrete is non-combustible, has a low thermal conductivity and has a low rate of temperature rise across a section. The cement paste in concrete undergoes an endothermic reaction when heated, which assists in reducing the temperature rise in fire-exposed concrete [6].

According to standard EN 13501-1, concrete fulfils the requirements of class A1 or A2 (no contribution to fire) because its mineral constituents are non-combustible and, in the majority of applications, can be describe as fireproof when properly designed. The mass of concrete may hold a large amount of heat. Also its porous structure provides a low rate of temperature rise across a section [6]. All this enables concrete to act as an effective fire shield.

Due to the low rate of increase of temperature through the cross section of a concrete element, internal zones do not reach the same high temperatures as a surface exposed to flames and this enables it to retain structural capacity and the temperature elevation of the reinforcement is delayed by the outer cover. Therefore, the loss of strength of the material is less significant, the load-bearing capacity of the member decreases slowly, and the behaviour in fire is much better than for steel and timber structures [1].

When concrete is exposed to the high temperature of a fire, a number of physical and chemical changes can take place and these physical changes are shown in Table 5 [13].

Table 5
Concrete in fire [13]

Temperature (°C)	What happens
1000	-
900	Air temperatures in fires rarely exceed this level but flame temperatures can rise to 1200°C and beyond.
800	-
700	-
600	Above this temperature, concrete is not functioning at its full structural capacity.
550-600	Cement based materials experience considerable creep and lose their loadbearing capacity.
400	-
300	Strength loss starts, but in reality only the first few centimetres of concrete exposed to a fire will get any hotter than this, and internally the temperature is well below this.
250-420	Some spalling may take place, with pieces of concrete breaking away from the surface.

Colour and surface damage of concrete at different temperatures are shown in Table 6 [1].

Table 6
Surface features of concrete at different temperatures [1]

Temp. (°C)	Colour	Cracks	Lost surface	Broken corners	Loose
900	Red	Wide, more, directionless	Lose after knocking	Every corner, different level	Serious, broken by finger (after cooling)
700	Dark red	Obvious, more	Less	Few	Obvious
500	Grey-white	Fine, more	Few	No	Slightly
300	Slightly white	Fine, few	Not yet	No	No
100	Same as normal temperature	No	No	No	No

During the heating process, the weight of the concrete specimen decreases gradually. When the temperature is in the range of 20-200°C, the concrete loses weight quickly, mainly because the free water in the specimen evaporates; when the temperature is in the range of 200-500°C, the weight is lost slowly as the chemically combined water in the cement mortar separates; when the temperature reaches 500°C, the calcium hydroxide (essential component produced from the hydration of cement) decomposes and dehydrates; when the temperature exceeds 600°C, the magnesium and calcium carbonates (in the aggregate) begin to decompose, so the aggregates become unstable and the weight loss may reach 10% and when the temperature is even higher, the outer layer of the concrete is damaged and spalls off, causing further weight loss [1]. Should be emphasized that dehydration and decarbonisation are endothermic reactions and they absorb some energy and therefore slow down heating [14].

The damage and failure phenomena in the concrete structures will appear successively for a long time: cracking and loosening on the surface, damage to the sides and corners, explosive spalling of the cover, reinforcement exposure, member deflection, gradual separation of the surface layers from the main body, damage area penetrating into the interior of the member, and, finally collapse of the entire structure may result [1].

Generally, the reinforced concretes have follows characteristics at elevated temperatures [1]:

1. Temperature distributed nouniformly in the interior

The temperature field in the structure varies continuously during the fire. The main factors determining the temperature field of the structure are itself temperature process, the shape and size of the members, and the thermal behaviour of the concrete material. Phenomena such as the internal forces, deformation and small cracks in the structure have less influence on the temperature field. On the contrary, the temperature field of the structure influences considerably the internal forces, deformation, and its bearing capacity.

2. Serious deterioration of material behaviour

At elevated temperatures, the values of the strength and elastic modulus of concrete and reinforcement decrease considerably and the deformation of both materials increases correspondingly. Besides that, as the temperature increases, the external damage of concrete (cracking, loosening, spalling) appears successively and gradually becomes more severe. As a consequence of that, serious reduction in the bearing capacity of structure and its members appears at elevated temperature.

3. Coupling effect of stress, strain, temperature, and time

For a structure at ambient temperatures it is necessary to study only the stress – strain relationship of the material. However, the strength and deformation of the material are strongly influenced by heating and in that case it is important for concrete to consider, except strength and strain, temperature and time also.

4. Redistribution of the stress on the member section and the internal forces of the structure

The nonuniform temperature field of the member section inevitably results in unequal temperature strain and stress redistribution on the section. The temperature deformation of the material at high temperature is restrained by the adjacent material at a different temperature, the joint and the support. So, the redistribution of internal forces (bending moment, shear force, axial force, and torque) of the structure is very important and different from that at room temperature.

5. Process and pattern of failure

Concrete structure under usual temperatures fails slowly with apparent signs. The structure and its members often fail suddenly at elevated temperatures because the deformation increases quickly, the failure duration is short, and fewer warning signs appear.

Concrete may protects against all the harmful effects of a fire and it is commonly used to provide stable compartmentation in large industrial and high buildings. By dividing these large buildings into compartments, the risk of total loss during a fire is in fact removed – the concrete floors and walls reduce the fire area both horizontally (through walls) and vertically (through floors).

The modest floor loads and relatively low temperatures experienced in most building fires mean that the loadbearing capacity of concrete is largely retained both during and after a fire [4].

3.1. Mechanical parameters

Compressive strength

The compressive strength of concretes cubes with different types of aggregate (L-limestone, G-granite) and two strength grades (20 and 40), at different temperature, are listed in Table 7 [1].

Table 7
Compressive strength of concrete cubes at elevated temperatures [1]

Concrete	20°C	60°C	100°C	150°C	200°C	300°C	400°C	500°C	700°C	900°C
C20L	30.5	28.3	28.2	29.0	31.1	32.5	30.6	24.7	10.6	3.6
C20G	28.8		26.1			30.3		22.8	8.3	2.4
C40L	55.0		50.3			56.7		43.7	21.4	5.0
C40G	54.1		48.2			54.3		40.9	13.8	2.9

When the temperature T is 100°C, the ratio between the compressive strength of concrete at elevated temperatures and at normal temperature, is between 0.88 and 0.94. The pressure of the water and vapour in the cracks and holes increases as the temperature rises causing tensile force in the surrounding solid materials. In addition, the stress concentration occurs at the tips of the cracks and accelerates the expansion of the cracks. Therefore, the compressive strength of the concrete reduces slightly.

When temperature is in the range 200–300°C, strength ratio is 0.95–1.08. In that moment the free water in the specimen is evaporated. The expansion coefficients of the coarse aggregate and the cement mortar of the concrete are not equal and due to the difference in thermal deformation between them cracks occur on the boundary of the aggregate and reduce the compressive strength of the concrete. On the other hand, as the combined water in the cement starts to release, it strengthens the adhesive action of the cement particles and relaxes the stress concentration at the crack tip, so it helps to increase the strength of the concrete. These contradictory factors act simultaneously, so that the compressive strength of the concrete first increases slightly and later reduces within the temperature range. The magnitude of amplitude of the strength depends on the quality of the raw materials, composite components, and mix of concrete [1].

When the temperature is equal 500°C, the compressive strength of the concrete decreases obviously and strength relationship is equal 0.75–0.85. The difference in thermal deformation between the aggregate and the cement mortar increases even more, and the cracks on the boundaries extend. Also, the water in chemical compounds of hydrated mortar is released with volume expansion, so the cracks expand and the compressive strength reduces more quickly.

When T ≥ 600°C, the quartz components in the unhydrated cement particles and the aggregates decompose and crystal is formed, accompanied by considerable expansion. As the temperature increases cracks also appear in the interior of some aggregates and expand. The compressive strength of concrete reduces sharply:

$$T = 700°C\ ,\ f_{Tcu}/f_{cu} = 0.30–0.50 \qquad (1)$$

$$T = 800°C\ ,\ f_{Tcu}/f_{cu} = 0.15–0.28 \qquad (2)$$

$$T = 900°C\ ,\ f_{Tcu}/f_{cu} = 0.05–0.12 \qquad (3)$$

According to above, the main reasons for strength loss can be summarized as follows:

- Crack and porosity form in the interior of concrete after water evaporates.
- The thermal behaviour of the coarse aggregate and the cement mortar are different, which causes a deformation difference and internal stress between them and results in cracking at their boundary.
- Coarse aggregate expands and cracks at high temperature. This internal damage in the concrete develops and accumulates continuously, and tends to be more serious as the temperature increases.

The testing device and the method, the shape and size of the specimen, the heating velocity, and the time the concrete sample was subjected to a defined temperature are different in each research experiment. Therefore, the experimental value (f_{Tcu}/f_{cu}) of concrete strength unavoidably shows a certain deviation. Various factors influence the compressive strength of concrete at elevated temperatures, and they are analysed according to the experimental results.

Strength grade of the concrete. The relative strength (f_{Tcu}/f_{cu}) of concrete (C20–C50) decreases with higher grades (Table 7), but there is less than 10% difference between them.

Type of aggregate. Concrete containing silicon aggregate (e.g., granite) has a slightly lower strength (f_{Tcu}/f_{cu}) compared with concrete containing calcium aggregate (e.g., limestone) at the same temperature. Concrete containing light-weight mineral aggregate has a much higher strength than concrete containing ordinary aggregate at high temperature [2].

Other factors. The larger the water/cement ratio or the water content of the concrete, the lower the strength (f_{Tcu}/f_{cu}) and the slower the heating velocity.

When a structure experiences an accidental high temperature and the fire eventually cools down to normal temperature, the analysis of results show that the interior of concrete is damaged when it is heated and a high temperature is maintained. When the concrete is cooling down, the temperature on its outer surface decreases quickly but the temperature in its interior remains high, so a nonuniform temperature field is formed and new damage occurs in the interior of the concrete. Some factors, such as the types of cement and aggregate, the water/cement ratio, and the age of the concrete, also influence the residual strength after cooling.

Figure 7 show the results obtained from specimens loaded to 40% of their compressive strength during the heating process and when the predefined temperature was reached, the specimens were loaded to failure. As can be seen, the compressive strength of concrete remains relatively unchanged up to 500°C. Above 500°C, the compressive strength of the concrete with siliceous aggregate starts to decrease rapidly and is considered ineffective at temperatures above 650°C, where the compressive strength has been reduced by approximately 50 percent of the value at normal temperatures. However, concrete with carbonate and lightweight aggregate behaves much better, its compressive strength remains relatively unchanged up to 650°C and is not considered to be ineffective until it reaches a temperature of 650°C [2].

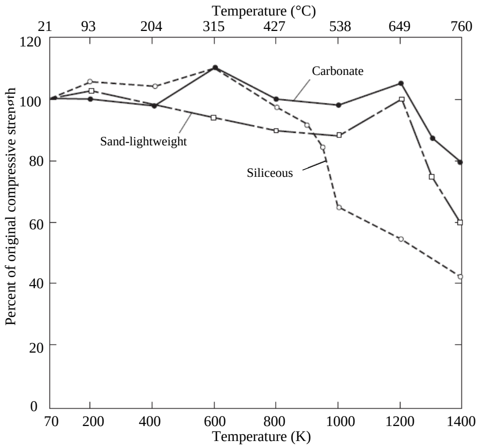

Figure 7. Strength-temperature relationships for carbonate, siliceous and sand-lightweight aggregate concretes [2]

Tensile strength

The relationships between tensile strength at high temperature and same strength at normal temperature (relative tensile strength) are listed in Table 8 [1].

Table 8
Tensile strength of concrete at elevated temperatures [1]

Temperature (°C)	f^T_t/f_t
100	0.78-0.90
300	0.66-0.88
500	0.52-0.60
700	0.24-0.32
900	-

The tensile strength of concrete reduces quickly when the temperature are in the range 20-100°C, but reduces slowly when the temperature between 100°C and 300°C, and linearly when above 300°C. When the temperature equal 900°C, the specimen approaches failure without further loading. The relative tensile strength is lower than the relative compressive strength which means that the internal damages in the concrete caused by thermal action has a stronger influence on its tensile strength.

Deformation

The main factors which influence on the thermal expansion strain of concrete are: the types of mineral components in the aggregates, the mix proportion and water content of the concrete, and the heating velocity of the specimen. In addition, differences in the testing methods and the measuring techniques cause significant deviation in the experimental data on the thermal strain of concrete.

When the temperature less than 200°C, the solid components of the concrete, such as the coarse aggregate and cement mortar, expand due to elevated temperatures and simultaneously shrink due to water loss. Both factors compensate and cause smaller strain. When the temperature equal 200°C, the thermal strain is $\varepsilon_{th} \approx (0.8–1.5)\times10^{-3}$.

When the temperature between 300°C and 600°C, the solid components expand as the temperature is elevated, and the cracks appear and extend on the boundary of the aggregate. Strain increasing very quickly. At T=500°C the thermal strain reaches a high value, e.g., $\varepsilon_{th} = (6–9)\times10^{-3}$.

When the temperature between 600°C and 700°C, the increasing rate of the thermal expansion strain slows down or even ceases. The expansion strain is obstructed possibly because the crystal of the mineral component within the aggregates varies and the internal damage in the concrete accumulates. At this time, the strain ε_{th} is greater than 10×10^{-3}.

During the heating process, the expansion strain (ε_{th}) of concrete is composed of four parts: heating expansion of the solid materials, shrinking due to water loss, appearance and extension of cracks on the boundary between the aggregate and cement mortar, and damage to the interior of the aggregates. However, during the cooling process, only the heating expansion of the solid materials may restore completely but the others remain unchanged.

Before loading of the specimen but during heating and maintaining the temperature, the initial strain and many cracks are formed in the interior of the concrete for various reasons. Some of these reasons are water evaporation, difference in thermal behaviour between the coarse aggregate and cement mortar, and the expansion and breaking of the aggregate [1].

Modulus of elasticity

The modulus of elasticity (E) of various concretes fall within a very wide range, from 5.0 GPa to 35.0 GPa, at room temperatures. It depends mainly on the water/cement ratio in the mixture, the age of concrete, the method of conditioning, and the amount and nature of the aggregates. Some authors found that the modulus of elasticity decreases rapidly with the rise of temperature, and does not depend significantly on the type of aggregate (Figure 8). However, from the other researches, the modulus of elasticity of concretes with normal-weight aggregate decreases faster with the rise of temperature than lightweight concretes [2].

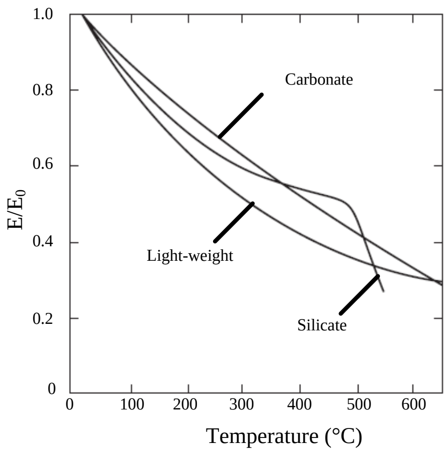

Figure 8. The effect of temperature on the modulus of elasticity of concretes with various type of aggregates [2]

3.2. Thermal parameters

In the case of concrete, the values of thermal parameters are dependent on the mix proportions. For example, values of specific heat for some type of concrete are given in Figure 9 [15].

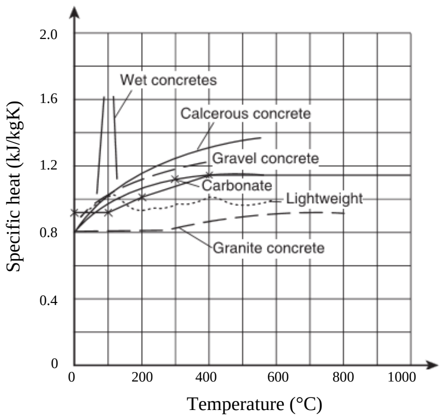

Figure 9. Variation of specific heat of concrete with temperature [15]

Thermal conductivity or coefficient of heat conduction

The conductivity of concrete decreases with rising temperature [9]. The coarse aggregate has the highest fraction of the total volume of concrete and has the predominant influence on its thermal behaviour. The coefficient of heat conduction of the aggregate depends mainly on the mineral composition, character of crystal, and structure of the granules.

The values of the coefficients of heat conduction of rocks are considerably different at normal temperature, but they tend to be similar at elevated temperature (when temperature is above 200°C). Also, they vary differently as the temperature increases. The coefficient of heat conduction of siliceous sandstone, dolomite, and limestone decreases quickly, at granite and gneiss decreases slowly, but at diabase and calcareous feldspar increases slowly when the temperature increases.

The water/cement ratio of the mortar during mixing has influence on the coefficient of heat conduction. A specimen with a large water/cement ratio contains more water and more microporosity is formed after the water is lost during hardening. This causes a decrease in the coefficient of heat conduction. The coefficient of heat conduction of concrete composed of an aggregate of different kinds of rock varies with temperature. The coefficient of ordinary concrete with siliceous aggregate is slightly higher than that of calcareous aggregate, but both reduce less as the temperature increases and are similar at very high temperature (>800°C) [1].

Coarse aggregate may be made of various porous mineral materials, such as pumice, slag, expanded clay, or shale. It may replace ordinary rock aggregate and be mixed to produce light-weight concrete. The granules of light-weight aggregate contain many interior pores, and considerably reduce the heat conduction. Therefore, the coefficient of heat conduction of light-weight concrete is far lower than that of ordinary concrete. The coefficient of heat conduction of concrete shows large variability and scatter because of the influence of various factors. The corresponding relationship between the coefficient of heat conduction and temperature, for concrete with different type of aggregate, is shown in Figure 10 [1].

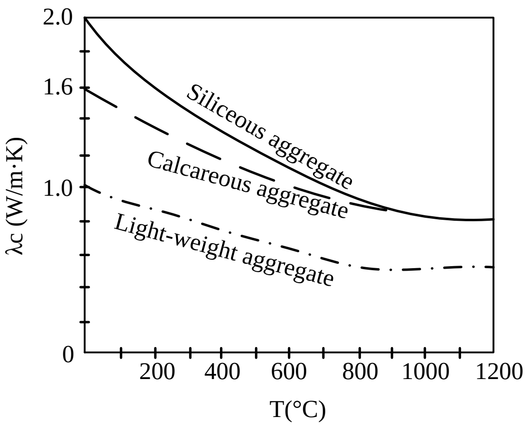

Figure 10. Calculated value of the coefficient of heat conduction of concrete [1]

This relationship given by another author is shown on Figure 11 [15]. The lower limit of thermal conductivity was obtained from comparisons with temperatures measures in fire tests on different types of concrete structures. The lower limit gives more realistic temperatures for concrete structures than the upper limit, which was obtained from tests on composite steel/concrete structures [14]. The concretes with normal-weight aggregate fall into a band with the values for lightweight concrete, but someone authors do not agree with that [15].

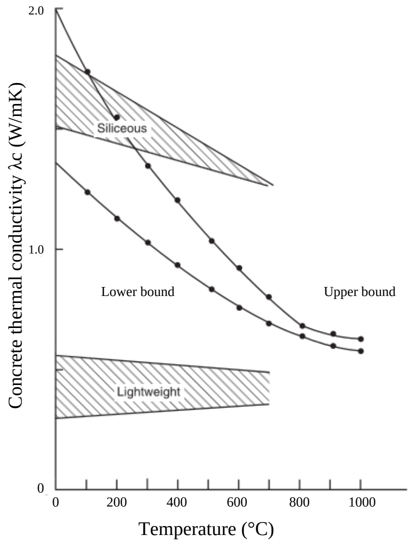

Figure 11. Variation of thermal conductivity of concrete with temperature [15]

Specific heat capacity (Cc)

The specific heat of dry concrete does not vary much with temperature. However, concrete structures always contain water which evaporates at temperatures above 100°C constituting a latent heat as the vaporization process consumes a lot of heat. Thus the specific heat capacity for normal weight concrete according to Eurocode 2 has a peak at temperatures 100°C and 200°C [9].

The type of aggregate has an influence on the specific heat capacity of concrete but not much. Specific heat capacity of concrete with siliceous aggregate (for example quartz) is slightly larger than that of calcareous aggregate (for example limestone), and that for various light-weight aggregates is slightly smaller than that of ordinary concrete. Other factors, such as the mixing ratio, water content, and age, have less influence on specific heat capacity [1].

Mass density (ρ_c)

The mass density is defined as the mass of the material per unit volume, and its units are kg/m^3. This material characteristic is also called the volume density. Changing of the concrete mass density during heating is shown on Figure 12.

During the initial period, mass density reduces obviously, because the water content evaporates. The solid components (aggregate and cement) expand after heating and the mass density decreases. This phenomenon exists throughout the heating process, and its influence increases gradually at high temperature.

Some types of rock aggregate with different mineral components have special properties at elevated temperature, which influence the mass density. For example, siliceous aggregate forms crystals at T = 600–800°C, and is accompanied by considerable volume expansion and sudden decrease in the mass density. Basalt and quartz are melted and sintered when T = 1200–1400°C, and then the mass density of the concrete increases suddenly. The mass density of concrete composed of various light-weight aggregates varies with temperature similar to that of ordinary concrete, but the amplitude of the variation is smaller (Figure 12).

However, to simplify the calculation, the mass density of concrete is normally taken as a constant (2200–2400 kg/m^3) irrespective of temperature.

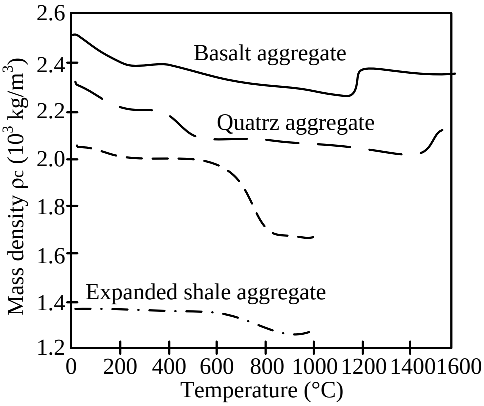

Figure 12. Mass density of concrete with different aggregates [1]

The basic thermal behaviour of concrete depends not only on the thermal behaviour of the coarse and fine aggregates and the hardened cement mortar but also the composition, water content, age, casting and compacting technique, and compactness of the concrete. Thus, the thermal parameters present large variation and scatter. If accurate thermal parameters are required for analysis, the specimens should be manufactured and tested specially and then the thermal parameters can be measured. The simplified values of the thermal parameters suggested in the relevant code can be used for practical engineering for the randomness and scatter of the temperature variation in a fire accident [1].

3.3. Fibre-reinforced concrete

Steel and polypropylene discontinuous fibres are the two most common fibres used in the concrete mix to improve structural properties of concrete. Studies have shown that polypropylene fibres in a concrete mix are quite effective in minimizing spalling in concrete under fire conditions. The polypropylene fibres melt at a relatively low temperature of about 170°C and create channels for the steam pressure in concrete to escape. This prevents the small explosions that cause the spalling of the concrete. The amount of polypropylene fibres needed to minimize spalling is about 0.1 to 0.25 percent (by volume). The addition of fibres improves certain mechanical properties at room temperature, such as tensile strength, ductility, and ultimate strain. However, there is very little information on the properties of this type of concrete during exposed to high temperature. Concrete with steel fibre (SFRC) exhibits, at elevated temperatures, mechanical properties that are more beneficial to fire resistance than those of plain concrete. The effect of temperature on the compressive strength for two types of concrete is shown in Figure 13 [2].

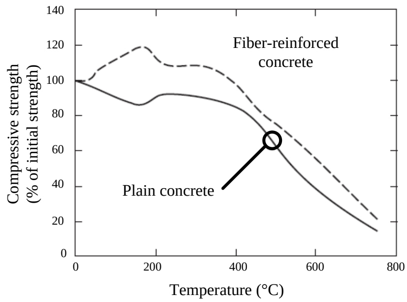

Figure 13. Effect of temperature on compressive strength of steel fibre-reinforced concrete [2]

As can be seen on figure, the strength of SFRC exceeds the initial strength of the concretes up to about 400°C. This is in contrast to the strength of plain concrete, which decreases slightly with temperatures up to 400°C. Above approximately 400°C, the strength of SFRC decreases at an accelerated rate.

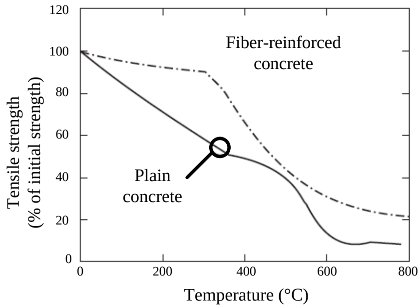

Figure 14. Effect of temperature on tensile strength of steel fibre-reinforced concrete [2]

The effect of temperature on the tensile strength of concretes with steel fibre is compared to that of plain concrete in Figure 14. The strength of SFRC decreases at a lower rate than that of plain concrete throughout the temperature range. Strength of steel-fibre concrete is significantly higher than that of plain concrete up to about 350°C. The increased tensile strength delays the propagation of cracks in fibre-reinforced concrete members and is highly beneficial when the member is subjected to bending stresses.

The type of aggregate has a significant influence on the tensile strength of concrete with steel fibre. The decrease in tensile strength for concrete with carbonate aggregate is higher than that for siliceous aggregate concrete. The thermal properties of SFRC, at elevated temperatures, are similar to those of plain concrete [2].

3.4. High-strength concrete (HSC)

The strength of concrete has significant influence on other properties of HSC. The material properties of HSC vary differently with temperature than those of NSC. This variation is more pronounced for mechanical properties, which is affected by these factors: compressive strength, moisture content, density, heating rate, percentage of additive, and porosity. The loss in compressive strength with temperature is higher for HSC than that for NSC, up to about 450°C. Figure 15 shows the comparison of strengths for NSC and HSC types, together with CEB and European design curves for NSC [2].

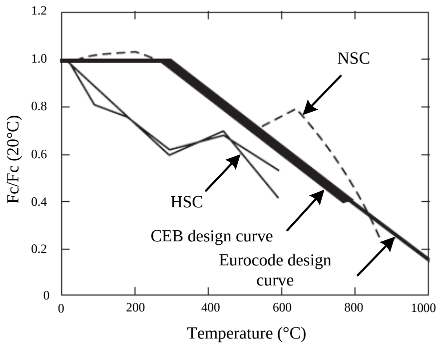

Figure 15. Comparison of design compressive strength and results of unstressed tests of HSC and NSC concrete [2]

The difference between compressive strength versus temperature relationships of concrete with normal-weight aggregate and concrete with light-weight aggregate is not significant. The variation of modulus of elasticity and tensile strength of HSC is similar to that of NSC with increasing the temperature.

The type of aggregate has significant influence on the thermal properties of HSC at elevated temperatures. Figure 16 shows the thermal conductivity of HSC, with siliceous and carbonate aggregates, as a function of temperature.

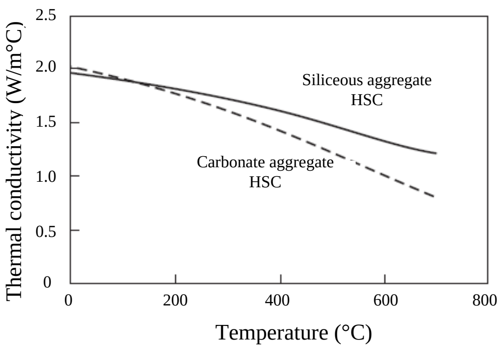

Figure 16. Thermal conductivity of high-strength concrete [2]

One of the major concerns for the use of HSC is regarding its behaviour in fire, in particular, the occurrence of spalling at elevated temperatures. The spalling in HSC can be minimized by creating pores through which water vapour can be relieved before vapour pressure reaches critical values. This is usually done by adding polypropylene fibres to the HSC [2].

3.5. Lightweight concrete

Lightweight concrete is usually made with normal cement and some form of lightweight mineral aggregate such as pumice or expanded clay or shale. Other possible materials to use include perlite and vermiculite. Lightweight concrete has been shown to have excellent fire resistance, due to its low thermal conductivity compared with normal weight concrete. Many lightweight aggregates have been manufactured at high temperatures, so they remain very stable during fire exposure [6].

4. STEEL

Steel is a material that is available in various product types: structural (hot rolled), reinforcing, prestressing or cold formed [10]. A steel structure is not combustible, however, when a steel structure is exposed to a fire, the steel temperatures increase and the strength and stiffness of the steel are reduced, leading to possible deformation and failure, depending on the applied loads and the support conditions. Heat conducts very quickly through steel [1]. An increase in steel temperatures depends on the severity of the fire, the area of steel exposed to the fire and the amount of applied fire protection [6]. The critical temperature of a steel member is the temperature at which it cannot safely support its load. The mechanical properties such as strength and modus of elasticity deteriorate in particular when the steel temperature exceeds 400°C (Eurocode 3, EN1993-1-2). Some building codes and structural engineering standard practice defines different critical temperatures which must not be exceeded when exposed to a standard fire exposure for a specified time. Steel structures must therefore usually be protected to reach a particular fire rating [9].

Full- scale tests and some real fires in large steel buildings have shown that well- designed steel structures can resist severe fires without collapse, even if some of the main load- bearing members are unprotected. Also thermal expansion of steel members can cause damage elsewhere in the building.
The main factors affecting the behaviour of steel structures in fire are:

- the elevated temperatures in the steel members;
- the fire limit state loads on the structure;
- the mechanical properties of the steel;
- the geometry and design of the structure.

The seriousness of the problem in structures exposed to high temperature depending most on four factors: (1) the function of the steel element, (2) its level of stress, (3) how the steel member is supported to other structural members, and (4) its surface area and thickness [16].

Equations for many properties of steel, as functions of temperature, are available in the Eurocode 3 and in many manual.

4.1. Mechanical parameters

Tensile strength

The structure of the material has a great influence on their behaviour in the fire. The structural steels and concrete-reinforcing bars have a randomly oriented grain structure, and their strength depends mainly on their carbon content. In the case of the prestressing steel, wires are usually made from high-carbon, pearlitic steels with an elongated grain structure, oriented in the direction of the cold work. In addition, light-gauge steel finds wide applications in lightweight framing and it is made from cold-formed steel. It can be used for structural members such as walls and floors. Mechanical properties of two typical steels (a structural steel and a prestressing wire) are presented in Figure 17 through Figure 19 [2].

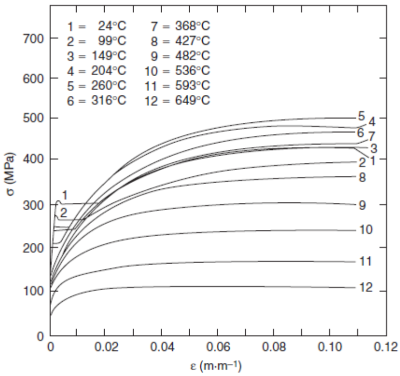

Figure 17. Stress-strain curves for a structural steel at room and elevated temperatures [2]

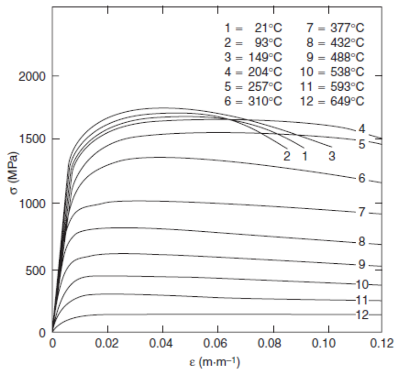

Figure 18. Stress-strain curves for prestressing steel at room and elevated temperatures [2]

Figure 17 and Figure 18 are stress-strain curves at room temperature (24°C and 21°C, respectively) and at a number of elevated temperature levels. Figure 19 shows the effect of temperature on the yield and ultimate strengths of the two steels. From figures can be seen that yield strength (σ_y) and modulus of elasticity (E) both decrease with increasing temperature, but the ultimate tensile strength (σ_u) increases slightly at moderate temperatures before decreasing at higher temperatures. From a structural viewpoint, the yield stress of steel is the most significant parameter in establishing its load-carrying capacity [16].

During fully developed fires, unprotected steel members often suffer large deformations, whereas well protected members usually exhibit little or no damage. For steel members exposed to fire but which remain straight after cooling in most cases no further assessment is necessary (Tide, 1998). The most structural steel do not suffer significant loss of strength when cooled after heating up to about 600°C while heating to higher temperatures can result in a strength reduction of up to 10% after cooling. The reduction in strength is much greater for high strength steels containing alloys such as vanadium and niobium. If necessary, some types of tests on specimens can be used to determine whether there has been a reduction in strength. Many types of high strength bolts have been heat-treated during manufacture which makes them susceptible to loss of strength after heating, in which case they should be replaced [6].

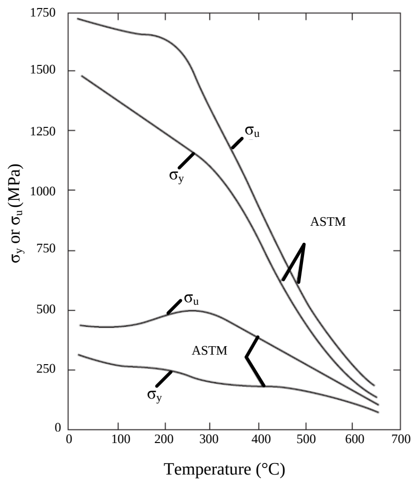

Figure 19. The ultimate and yield strength for a structural steel (ASTM A36) and a prestressing steel (ASTM A421) at elevated temperatures [2]

High strength steels, especially prestressing, are much more sensitive to strength loss if they are heated to temperatures above 400°C. In the case of prestressing steels, cooling after heating to 500°C can have a 30% loss of strength while heating to 600 °C can result in a 50% loss of strength (Gustaferro and Martin, 1988) [6].

Deformation

Creep is a very structure-sensitive property and may show a substantial spread. This parameter is relatively insignificant in structural steel at normal temperatures. However, it becomes very significant at temperatures over 400°C or 500°C and is highly dependent on stress level. At higher temperatures the creep deformations in steel can accelerate rapidly, leading to plastic behaviour and 'runaway' failure [6].

Modulus of elasticity

For a variety of common steels, the modulus of elasticity (E) is about 210 GPa at room temperature. Figure 20 shows its variation with temperature for structural steels and steel reinforcing bars (E_0 is the modulus of elasticity at room temperature).

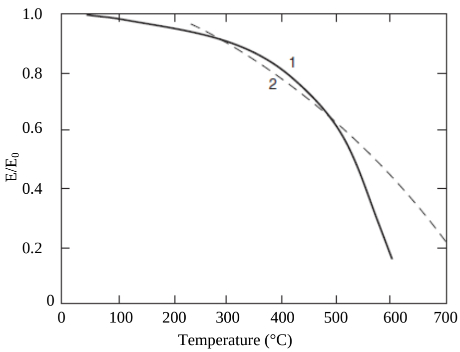

Figure 20. The effect of temperature on the modulus of elasticity of structural steels (1) and reinforcing bars (2) [2]

4.2. Thermal parameters

All steel properties, except density (about 7850 kg/m^3), are strongly influenced by temperature. For any calculation, it is necessary to know the thermal properties of the materials [6].

Thermal conductivity

The thermal conductivity (λ) of steel is not easy to define. For carbon steels it usually varies within the range of 46 W/mK to 65 W/mK [2]. Standard EN 1993-1-2 differs two range of temperatures for calculation of λ:

- $20°C \le \theta \le 800°C$
- $\theta \ge 800°C$.

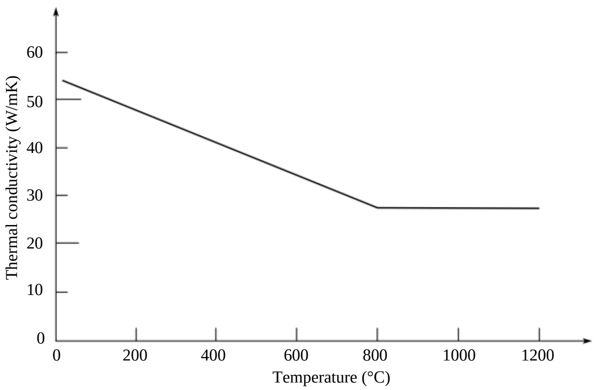

Figure 21. Thermal conductivity of steel as a function of temperature [6]

The thermal conductivity of steel varies according to temperature as shown in Figure 21 [6] and it is in accordance with a general rule of physics inversely proportional to the molecular weight [9]. As seen on figure, it is reducing linearly from 54 W/mK at 0°C to 27.3 W/mK at 800°C (CEN, 2005b). For simple calculations the thermal conductivity λ (W/mK) can be taken as 45 W/mK [6].

The heat conductivity of carbon steel is in the order of 30 times higher than the corresponding value for concrete and 100–1000 times higher than that of insulation products. The higher purity of a metal, the better it conducts heat. Thus contents of carbon and alloying metals such as chrome reduce the conductivity, and consequently stainless steel is a relatively poor conductor [9].

Specific heat flux

The specific heat capacity is usually a more significant parameter than the conductivity for the development of temperature in fire-exposed steel structures [9]. Its change depends on the temperature as is shown in the Figure 22 [6].

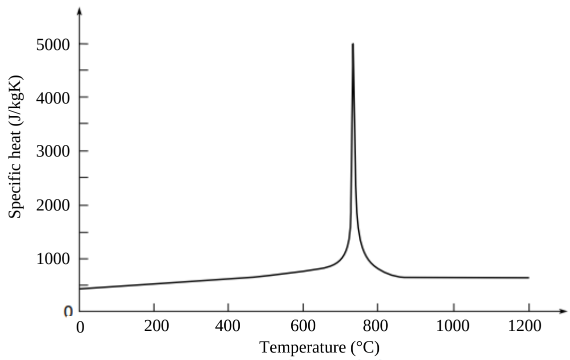

Figure 22. Specific heat of steel as a function of temperature [6]

In this situation several areas of temperature can be distinguished:

- 20°C ≤ θ ≤ 600°C
- 600°C ≤ θ ≤ 735°C
- 735°C ≤ θ ≤ 900°C
- 900°C ≤ θ ≤ 1200°C

As can be seen metallurgical change occurs at 735ºC due to phase changes of the steel [9]. For simple calculation constant value of 0.6 kJ/kgK may be taken [6] but a value of 0.46 kJ/kgK is recommended which normally yields calculated temperatures on the safe side (overvalued) [9]. However, for more accurate calculations are recommended Eurocode.

Thermal expansion

Coefficient of thermal expansion (β) of steel is a structure-insensitive property. The testing reveals substantial contraction of the material at about 700°C, which is associated with the transformation of the ferrite-pearlite structure into austenite which is associated with a change in behaviour of material.

The coefficient of thermal expansion is usually taken to be 11.7×10–6/°C, at room temperatures. At higher temperatures such as those experienced in fires, the coefficient increases, and a discontinuity occurs between 700°C and 800°C. For normal design purposes, Eurocode 3 Part 1.2 (CEN, 2005b) recommends a linear coefficient of 14.0×10–6/°C [6].

Creep strain

Creep is relatively insignificant in structural steel at normal temperatures. However, it becomes very significant at temperatures over 400 or 500°C. Many experiments on the creep behaviour of steel at elevated temperatures show that the creep is highly dependent on temperature and stress level [6].

Critical temperature

The critical temperature of steel is often used as a bench mark for determining the failure of structural members exposed to fire. This ensures that the yield strength is not reduced to less than that of 50 percent of value at ambient temperature. The critical temperature for various types of steels is given in Table 9 [2].

Table 9
Critical temperature for various types of steel [2]

Type of steel	Standard/Reference	Temperature (°C)
Structural steel	ASTM	538
Reinforcing steel	ASTM	593
Prestressing steel	ASTM	426
Light-gauge steel	EC 3	350

4.3. Protected systems

There are many methods of protecting steel members from the influence of fire. Structural steel buildings with applied fire protection can be designed to have excellent fire resistance. Unprotected steel structures tend to perform poorly in fires compared with for example reinforced concrete or heavy timber structures, because the steel members are usually much thinner and steel has a higher thermal conductivity than most other materials. Unprotected steel structures can survive some fires if the severity is low and the steel does not get too hot.

One example for protected steel beam with insulating material thickness of 15mm and 30mm and unprotected steel beam is displayed in Figure 23 [6].

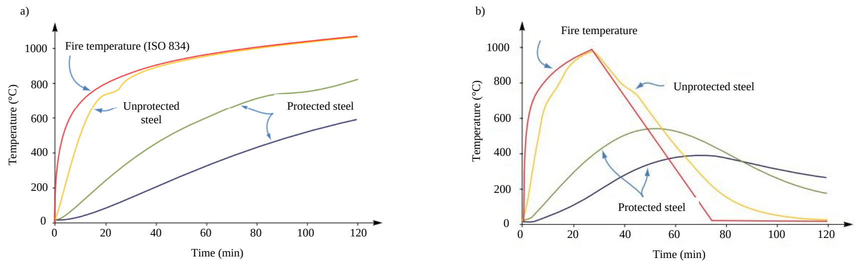

Figure 23. Typical steel temperatures for unprotected and protected steel beams exposed to: a) the standard fire; b) a parametric fire [6]

There are many alternative passive fire protection systems to reduce the rate of temperature increase in steel structures exposed to fire [6]:

- Concrete encasement
- Board systems
- Spray-on system
- Intumescent paint
- Protection with timber
- Concrete filling.

One of common form of fire insulation applied on steel structural members to achieve required fire resistance is spray applied fire resistive materials (SFRM), which work by delaying temperature rise in steel. SFRM is mainly composed of base materials such as gypsum, cementitious and mineral fibre and other additives such as vermiculite [4].

5. REINFORCEMENT

5.1. Mechanical parameters

The reinforcing steel is very sensitive to temperature increase and its strength changes significantly with temperature, and must be taken into account in any structural calculation [2].

The mechanical behaviour mainly depends on the composition of the alloys and production technology, including thermal treatment. The crystal structure of steel transforms at elevated temperatures and this causes the corresponding variations in its mechanical behaviour. The experimental data provided from testing reinforcement at elevated temperatures show that the strength and deformation behaviour deteriorate gradually as the temperature increases.

At temperature of 800°C, the strength for all strength grades of reinforcement reach much lowers value, which is only about 10% of that at normal temperature. The strength and deformation behaviour of reinforcement vary under different temperature–stress paths, and this is revealed experimentally. However, the experimental data and theoretical analysis available are still not sufficient agreed and more comprehensive experimental investigations are needed [1].

5.2. Thermal parameters

The existing reinforcement has little influence on the temperature distribution in the interior of a structure under fire conditions because it generally makes up only a small percentage (<3%) of the total volume. When the temperature field of a structure is analysed, the structure is assumed to be homogeneous and the reinforcement can be ignored. The main constituents of the steel used in building structures are iron and carbon, and the steel is divided into low, medium, and high carbon-steels according to the carbon content [1].

The different element components, their content in the steel and heat-treatment processes of the steel influence the indices of its thermal behaviour. Pure iron has the highest coefficient of conduction and it decreases gradually as the content of carbon and alloy in the steel increases. The coefficients of heat conduction of carbon steel and low-alloy steel decrease monotonically as the temperature increases, but the variable rates are reduced gradually. However, some steels containing more alloy and the coefficient of heat conduction increases slowly with temperature. Because iron, carbon, and other alloy elements have different values of specific heat capacity, the steels composed of different types and content of alloy elements have corresponding values of heat capacity (Cs). The value increases slightly and gradually with temperature, but the variation is small.

The mass density (ρ_s) of steel also varies slightly because of the different types and content of alloy elements in steel. The mass density of pure iron is high and reaches 7871 kg/m^3; for carbon and low-alloy steels, the mass density is 7850 kg/m^3. The volume of the steel expands and the mass density decreases slightly as the temperature increases, but it is generally taken as a constant during analysis of the temperature field. General variable ranges for the thermal parameters of steel are given in Table 10 and compared with concrete.

Comparing the data listed in Table 10, it is found that steel is a good heat conductor and concrete is not. The ratio of the coefficients of heat conduction is enormous. The mass heat capacity of steel is obviously smaller than that of concrete, because it is defined by the mass of the material. This thermal behaviour and the values of the parameters for concrete and steel have an obvious impact on the value and distribution of temperature in the structure at elevated temperatures.

Table 10
General range of thermal parameters of steel and concrete [1]

Material	Coefficient of heat conduction λ (W/m·K)	Mass heat capacity C (kJ/kg·K)	Mass density ρ (kg/m^3)
Steel	55-28	0.42-0.84	7850
Concrete	1.6-0.6	0.84-1.26	2300

6. MASONRY

6.1. Mechanical parameters

Compressive strength

Compressive strength of brick varies in a very wide range, from 9 MPa to 110 MPa. This value is an order of magnitude greater than the stresses allowed in the design of grouted brickwork. Since brick is rarely considered for important load-bearing roles in buildings, there has been little interest in the mechanical properties of bricks at elevated temperatures.

Ceramic clay bricks lose very little strength after heating to temperatures of 1000°C, but the mortar usually suffered damages. In the case of reinforced concrete masonry, it will need to be assessed in the same way as normal reinforced concrete [6].

Modulus of elasticity

The modulus of elasticity of brick (E) is usually between 10 GPa and 20 GPa [2].

6.2. Thermal parameters

Thermal conductivity

Thermal conductivity of masonry is dependent on density and the temperature, and values are given in Figure 24 [15].

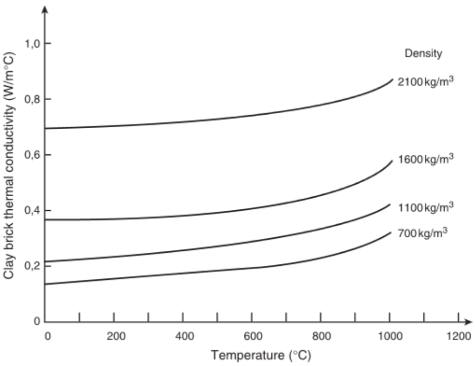

Figure 24. Variation of the thermal conductivity of masonry with temperature [15]

Specific heat

Specific heat is also dependent on temperature of the masonry and values are given in Figure 25 [15].

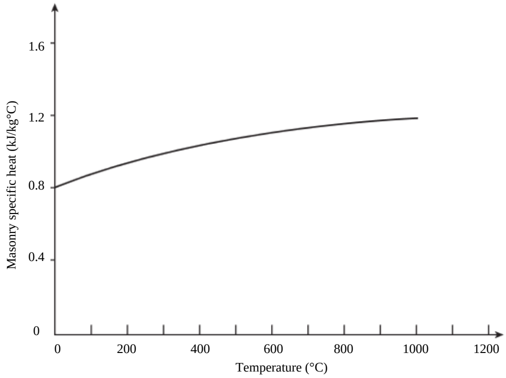

Figure 25. Variation of the specific heat of masonry with temperature [15]

Thermal expansion

At room temperature, the coefficient of thermal expansion for clay bricks is about $5.5\cdot10^{-6}$ m/mK [2].

Mass density

The density (ρ) of bricks ranges from 1660 kg/m^3 to 2270 kg/m^3, depending on the raw materials used in the manufacture and on the production technique. The true density of the material (ρ_t) is somewhere between 2600 kg/m^3 and 2800 kg/m^3 [2].

7. TIMBER

Wood is an inhomogeneous material which properties vary with the direction in which the measurement is made. It is a complex mixture of natural polymers of high molecular weight and the most important are cellulose (~50%), hemicellulose (~25%) and lignin (~25%) although these proportions vary from species to species [17].

Wood is widely used as structural members in low-rise constructions [2]. However, timber is combustible, cannot prevent a fire, and can even enhance a fire after it has started [1]. Therefore, fire damage to exposed timber surfaces is immediately visible.

The complexity of wood makes it difficult to interpret the burning behaviour. Because of the grain structure, properties vary with direction: thus the thermal conductivity parallel to the grain is about twice that perpendicular to the grain, and there is an even greater difference in gas permeability (of the order of 103). Volatiles generated just below the surface of the unaffected wood can escape more easily along the grain than at right angles towards the surface [17].

At temperatures above 200–250°C, wood discolours and chars, although prolonged heating at lower temperatures (≥120°C) will have the same effect. The physical structure begins to break down rapidly at temperatures above 300°C. In that moment small cracks appear in the char, perpendicular to the direction of the grain. The cracks will gradually widen as the depth of char increases and permits volatiles to escape easily through the surface from the affected layer. The burning of wood is a very complex process which will be more complicated by the presence of the layer of char and also the interactions within the hot char. Even during active burning, small quantities of oxygen may diffuse to the surface and reacts releasing heat that would contribute to the decomposition of the virgin wood under the layer of char. There is a wide variation in the composition and structure between woods of different species [17].

Heavy timber structural members such as beams, columns or floors will be charred on the surface with undamaged wood inside. The residual wood under the charred layer can be assumed to have unchanged strength and its size can be determined by scraping away the changed part on the surface.

Fire- exposed heavy timber members tend to deform much less than unprotected steel members. Fire- damaged timber members do not need to be replaced if the residual cross section has sufficient strength to carry the design loads. For future it may be necessary to apply additional protection such as new layers of wood or gypsum board. In the case of severely damaged members it will need to be replaced.

Light timber frame structures are protected from fire by linings of non- combustible material (for example gypsum board). After a severe fire the linings will certainly be damaged or some linings may have fallen off due to the effects of the fire or fire- fighting activities. All damaged linings should be removed to inspect damage on members, to the studs or joists. Any charred timber will have reduced load capacity and for calculations will be necessary to assess the strength of the residual members [6].

7.1. Mechanical parameters

Wood is an orthotropic material meaning the strength and stiffness in longitudinal and transverse directions are different and influenced by grain orientation. The mechanical properties of wood depend on moisture content, rate of charring, and grain orientation. Regardless of the type of wood, mechanical properties are related and roughly proportional to the density [2]. There is very little information on stress-strain relationships for wood.

Deformation

The coefficient of linear thermal expansion (β) ranges from 3.2×10^{-6} m/mK to 4.6×10^{-6} m/mK along the grain, and from 21.6×10^{-6} m/mK to 39.4×10^{-6} m/mK across the grain. Wood shrinks at temperatures above 100°C, because of the reduction in moisture content. Some research reported that the amount of shrinkage can be estimated as 8% in the radial direction, 12% in tangential direction, and an average of 0.1% to 0.2% in the longitudinal direction [2].

Creep in wood is complicated by changes in moisture content such that creep deformations tend to be larger in environments where the moisture content of the wood fluctuates over time, hence creep can become a major concern in fire - exposed wood which is at temperatures around 100°C [6].

Modulus of elasticity

The modulus of elasticity (E) of air-dry, clear wood, along the grain varies from 5.5 GPa to 15.0 GPa, and its compressive strength (σ_u) varies from 13 MPa to 70 MPa.

Figure 26 shows the changes of the modulus of elasticity and compressive strength of oven-dry, clear wood with temperature. E_0 and $(\sigma_u)_0$ in the figure are modulus of elasticity and compressive strength at room temperature, respectively. The modulus of elasticity decreases slowly with temperature up to about 200°C, when it reaches about 80 percent, and then the decline is more rapid. The compressive strength also drops linearly to about 80 percent at about 200°C, and then the drop is more rapid—to about 20 percent around 280°C [2].

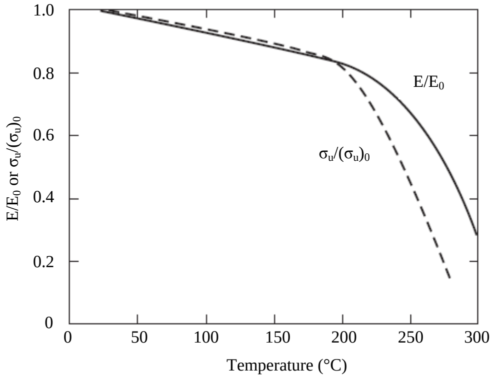

Figure 26. The effect of temperature on the modulus of elasticity and compressive strength of wood [2]

The tensile strength exhibits behaviour similar to that of compressive strength, but the decline in tensile strength is less rapid with temperature.

The moisture content plays a significant role in determining the strength and stiffness, so increasing moisture content leading to higher reduction. The formulas for reduced stiffness and design strength can be found in Eurocode 5 (Part 1.2) [2].

7.2. Thermal parameters

Thermal conductivity and specific heat

The thermal conductivity of timber is dependent of gravity and moisture percent in the timber, while the specific heat is dependent of temperature of the wood [2]. For example, the thermal conductivity across the grain of some type of pine was measured from 0.86 W/mK to 0.107 W/mK, between room temperature and 140°C. The thermal conductivity increases initially up to a temperature range of 150°C to 200°C, then decreases linearly up to 350°C, and finally increases again beyond 350°C. Figure 27 shows the apparent specific heat for the same pine, as a function of temperature. The accuracy of the curve is somewhat questionable, however, it provides useful information on the nature of decomposition reactions that take place between 150°C and 370°C [15].

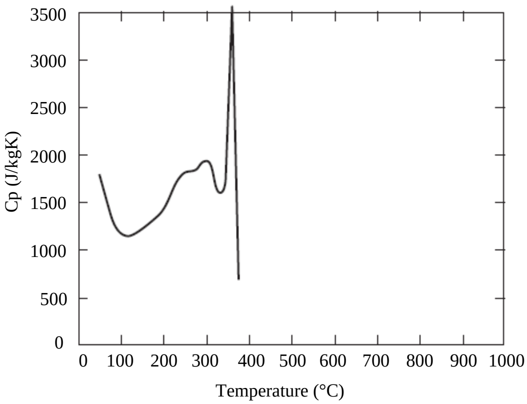

Figure 27. Specific heat for a pine of 400 kg/m3 [15]

Mass density

The oven-dry density (ρ) of woods ranges from 300 kg/m^3 to 700 kg/m^3. The density of wood decreases with temperature while the density ratio (ratio of density at room temperature to that at elevated temperature) drops to about 0.9 at 200°C and then declines sharply to about 0.2 at about 350°C [2].

Charring

The rate of burning of wood is frequently reported as the 'rate of charring' (mm/min) [17]. Charring is one of the main properties of wood associated with high temperature and should be considered in predicting performance under fire conditions. The rate of charring is influenced by the radiant heat flux or the fire severity. Generally, a constant transverse-to grain char rate of 0.6 mm/min can be used for woods subjected to standard fire exposure while the charring rate parallel to the grain of wood is approximately twice of that. These charring rates should be used only when attempting to model the performance of wood sections in the fire resistance furnace.

Charring is influenced by a number of parameters, the most important ones being density, moisture content, and contraction of wood. It is reasonable to modify the 0.6 mm/min to approximately 0.4 mm/min for moist dense wood, or to 0.8 mm/min for dry and light wood. Charring rates for different types of wood can be found in literatures. Eurocode7 gives an expression for charring depth in a wood member exposed to standard fire [2].

Char can insulate the material below it from the heat source because char is generally less conductive than the wood from which it originates. This phenomenon complicates determination of a "charring rate" because this rate changes over time. It is possible to determine an average charring rate for substances exposed to a calibrated energy source such as a furnace, but for real fires it is not the case. The charring rate of most common woods has been found to range between 0.5 mm/min and 0.8 mm/min [18].

8. ALUMINIUM

This type of material has the lower softening and melting temperature compared with steel. The value of thermal conductivity is between 180 W/m°C and 240 W/m°C [15] but it is different for different alloys [19].

However, the limiting temperature for aluminium is around 200°C as above this temperature the strength loss is such that any factor of safety in the design is completely eroded. The limiting temperature is taken as a function of the exact aluminium alloy in use as the temperature-related strength loss is very dependent on the amounts and type of alloying constituents. The density used in calculations may be taken as 2700 kg/m^3 [19]. According to data in EN 1999-1-2, the specific heat of aluminium varies from 913 J/kg°C at room temperature (20°C) to 1108 J/kg°C at elevated temperature 500°C, it is 2.1 and 1.7 times higher than that of steel, respectively [19].

9. PLASTICS AND PLASTIC-BASED COMPOSITES

Polymers are macromolecules that have high molecular weights, whose individual molecules consist of long "chains" of repeated units which are derived from simple molecules known as monomers [17]. Most polymers are based on carbon known as organic polymers. High molecular weights mean that polymer can exist in solid or liquid form, but are too long to be volatile [4]. Simple molecules may be bonded together in a line, a sheet, or a three-dimensional matrix. Most polymers have significant fractions of other ingredients, added to give them desirable properties, such as fillers or plasticizers. Some polymers can contain up to 50% plasticizer which make polymers soft and pliable [18].

With respect to flammability, the yield of volatiles from the thermal decomposition of a polymer is much less for highly cross-linked structures since much of the material forms an involatile char, thus effectively reducing the potential supply of gaseous fuel to a flame [17]. It is necessary to understand the stages in the conversion of long molecular chains into volatile fragments. This is often referred to as "pyrolysis" or "gasification", but these terms encompass a complex set of chemical and physical processes, leading to the production of volatile flammable molecules. Both the physics and chemistry of polymers affect their thermal decomposition and burning behaviour. Depending on their thermal history, most polymers exert a degree of crystallinity, giving a sharper transition between solid and liquid phases. The chemical composition of the molecular chains exerts a profound influence on the thermal decomposition of the polymers, with chain branching, double bonds, or oxygen in the polymer backbone reducing the thermal stability, and aromatic rings and crosslinking of the polymer backbone increasing the thermal stability [4].

Synthetic polymers are usually classified into two main groups, thermoplastics and thermosetting resins. As a third group can be taken the elastomers that may be distinguished on the basis of their rubber-like properties [17]. The main difference between thermoplastics and thermosetting polymers is that the latter are cross-linked structures that will not melt when heated. However, at a sufficiently high temperature, many decompose to give volatiles directly from the solid, leaving behind a carbonaceous residue. On the other hand, the thermoplastics will soften and melt when heated, which will modify their behaviour under fire conditions [17]. Thermosets are generally stronger, but more brittle than thermoplastics, have higher thermal stability, higher dimensional stability, higher rigidity, and resistance to creep and deformation under load [4].

One type of polymer are plastic materials. Plastics and especially plastic-based composites are structurally very efficient in terms of their weight strength ratio but poor when exposed to the effects of fire. This therefore means that such materials need extensive levels of protection in order to retain load-carrying capacity at elevated temperatures. This therefore means that the insulation thicknesses need to ensure that the temperatures within the plastic element need to be kept close to ambient. There is an additional problem in that some plastics decompose with temperature and emit highly inflammatory or toxic gases.

The characteristics of some the most popular thermoplastics polymers are shown below [2]:

Polyethylene (PE): Polyethylene begins to cross-link at 202°C and to decompose (reductions in molecular weight) at 292°C though extensive weight loss is not observed below 372°. The products of decomposition include a wide range of alkanes and alkenes. The major products of decomposition are propane, propene, ethane, ethane, butane, hexene-1, and butene-1. A broad range of activation energies has been reported, depending on the percent conversion and the initial molecular weight. Decomposition is strongly enhanced by the presence of oxygen, with significant effects detectable at 150°C.

Polypropylene (PP): The stability of polypropylene is lower as compared to polyethylene. As with polyethylene, chain scission and chain transfer reactions are important during decomposition. Reductions in molecular weight are first observed at 227°C to 247°C and volatilization becomes significant above 302°C. Oxygen drastically affects both the mechanism and rate of decomposition. The decomposition temperature is reduced by about 70°C, and the products of oxidative decomposition include mainly ketones. At temperatures below the melting point, polypropylene is more resistant to oxidative pyrolysis as oxygen diffusion into the material is inhibited by the higher density and crystallinity of polypropylene.

Polyvinylchloride (PVC): The most common halogenated polymer is PVC. It is one of the three most widely used polymers in the world, with polyethylene and polypropylene. Between 227°C and 277°C, hydrogen chloride gas is evolved. It is very important to point out that the temperature at which hydrogen chloride starts being evolved in any measurable way is heavily dependent on the stabilization package used. The rate of changing depends on the molecular weight, crystallinity, presence of oxygen, hydrogen chloride gas, and stabilizers.

Polymer products are generally low in cost, have light weight, available in a wide range of colour, low thermal and electrical conductivity, good toughness and good resistance to acids, bases and moisture [4].

10. GYPSUM

Gypsum products are used extensively in the building industry in the form of boards. Gypsum board is produced by mixing water with "plaster of paris" (calcium sulphate hemihydrate: $CaSO_4{\cdot}0.5H_2O$). The interlocking crystals of $CaSO_4{\cdot}2H2O$ are responsible for the hardening of the material. The core of the boards is fabricated with "plaster of paris", into which additives are mixed [2].

There might be significant variation in fire performance of the gypsum board based on the type and the formulation of the core, which varies from one manufacturer to another. Gypsum is an ideal fire protection material. The water inside the gypsum plays a major role in defining its thermal properties and response to fire. On heating, it will lose the two H_2O molecules at temperatures between 125°C and 200°C. The heat of complete dehydration is

0.61-106J per kg gypsum. Due to the substantial absorption of energy in the dehydration process, a gypsum layer applied to the surface of a building element is capable of markedly delaying the penetration of heat into the underlying loadbearing construction.

The thermal properties of the gypsum board vary depending on the composition of the core. The thermal conductivity of gypsum products is difficult to assess, owing to large variations in their porosities and the nature of the aggregates. A typical value for plaster boards of about 700 kg/m^3 density is 0.25 W/mK. The coefficient of thermal expansion of gypsum products may vary between 11.0×10^{-6} m/mK and 17×10^{-6} m/mK at room temperature, depending on the nature and amount of aggregates used [2].

There is not much information about the mechanical properties of the gypsum board at elevated temperatures because these properties are difficult to obtain experimentally. The strength of gypsum board at an elevated temperature is very small and can be neglected.

Inspection of gypsum board can give an indication of the duration of fire. When gypsum board is exposed to fire it dehydrates steadily from the hot surface. The depth of dehydration can be observed by breaking a small piece of board to locate the transition between the soft dehydrated plaster and the solid gypsum of the original board. Typical gypsum board dehydrates at approximately 0.5mm per minute [6].

11. GLASS

Glass is a vitreous solid material with crystal structure similar to a liquid. During the heating, it goes through a decreasing viscosity.

Glass is sometimes used in fire resisting barriers, where it can only provide an integrity rating, because it has no structural capability at elevated temperatures and cannot provide an insulation rating unless it is coated with some sort of intumescent coating. If glazing is to be used in a fire resisting barrier, it must be assembled with special glasses, either wired glass (reinforced with fine wires in both directions) or specially formulated fire resistant glass. Fire resisting glazing is usually installed in steel frames which clamp the glass and prevent it from deforming excessively when it gets hot. Aluminium frames cannot be used because of low melting temperatures. Glazed assemblies can be tested in full- scale fire resistance tests, but the assessment is only for the integrity criterion.

Most typical glass softens or melts in the temperature range 600°C to 800°C, but it will crack or break if exposed to thermal shock at much lower temperatures, due to differential temperatures within the glass or because of expansion of the surrounding frame. Normal window glass is assumed to break and fall out of the windows at the time of flashover (typically around temperatures of 500–600°C), although tests have shown that this does not always occur. Toughened glass or heat strengthened glass may not shatter at high temperatures. Double glazing tends to remain in place much longer than single layers of glass.

A number of proprietary insulated glazing systems have recently been developed, consisting of alternating layers of glass or sodium silicate with transparent intumescent materials. These products are transparent at room temperatures, but become opaque at high temperatures, achieving fire resistance of up to 2 h. Glass walls and windows can provide resistance to fire spread if they are sprayed continuously with water from a properly design sprinkler system (Kim et al., 1998; England et al., 2000) [6].

12. FRP – FIBRE REINFORCED POLYMERS

Strengthening by means of fibre reinforced polymers (FRP) is very popular primarily because of its high mechanical properties and relatively low cost. FRP composites are composed of high strength fibres (e.g. carbon, glass, aramid) and thermosetting organic matrices usually epoxy resin for bonding it to structure. The fibres carry the tensile forces, whereas the matrix transfers the stress to the concrete support. They are easy to install, have a high strength-to-weight ratio, and have suitable mechanical properties. FRP composites can be externally bonded (ER) to the element surface or placed within groove carved into the element and filled with organic matrices (Near Surface Mounted technique, NSM).

When exposed to high temperature, the problem arises when the glass transition temperature, Tg, of the polymer matrix is achieved, due to the softening of the resin, which reduces the capacity of transfer of forces between the fibres. The precise definition of the value of Tg is a problem for identification, because the progressive nature of the softening process. FRPs which polymerize in situ applications are characterized by very low Tg (between 45°C and 80°C for normal and heat resistant resins, respectively). When FRPs are using as internal reinforcement, is possible to obtain, Tg above 100°C [20].

Although overcoming the Tg implies a reduction in strength of the reinforcement, the significant degradation is reached at temperatures close to melting of the resin (temperature of crystallization, Tc > Tg) or even higher. Experimental studies showed that the softening of the resin which begins at Tg, involves a drastic reduction of the adhesion properties and the efficiency of the strengthening system for existing structures, which mainly depends on the effectiveness of the bond between FRP and concrete, is strongly affected by the temperature [20]. FRP are classified as anisotropic materials therefore possess varying properties in different directions [4].

The comparison between steel and FRP strengthening systems showed that FRP, in particular sheets, without protection behave better than steel plates because of the lower heat conductivity and their smaller weight. Using FRP as externally strengthened for RC beams, walls or slabs need the protection with additional insulation in order to avoid the debonding between FRP sheets or laminates and concrete support [20].

13. INSULATION FOR PROTECTION

Insulation material is often used as a fire protection material for structural members such as columns, beams, floors and walls. The insulation helps delay the temperature rise of structural members, thereby enhancing fire resistance. There are a number of insulation materials, but mineral wool and glass fibre are the two most widely used insulation materials in walls and floors. Other insulation materials used for fire protection include intumescent paints, spray mineral fibres, insulation boards, and compressed fibre board. The thermal properties of insulation play an important role in determining the fire resistance.

There is not much information available on the thermal properties of various types of insulation. The differences in thermal conductivity values at higher temperatures are mainly due to variation in the chemical composition of fibre. Fire resistance tests on walls and floors have shown that the mineral fibre insulation performs better than glass fibre insulation. This is mainly because glass fibre melts in the temperature range of 700–800°C while the melting point for mineral fibre insulation is higher. The density of glass fibre is about 10 kg/m^3 and is much lower than that of rock fibre, which is about 33 kg/m^3 [4].

Also the moisture content of the insulation material has an effect on the thermal properties. The thermal properties for fire insulation are shown at room temperature and they can vary significantly with temperature and also with insulation composition [4].

14. EUROPEAN STANDARD EN 13501-1

European Standard EN 13501-1 [21] provides the reaction to fire classification for all products and building elements. According to this Standard, reaction to fire is the response of a product in contributing by its own decomposition to a fire to which it is exposed, under specified conditions.

Products are considered in relation to their end use application are divided into three main categories:

- construction products,
- flooring,
- linear pipe thermal insulation products.

Construction products are classified according to harmonized test methods in Euroclasses A1, A2, B, C, D, E and F. Products classified in a given class are deemed to satisfy all the requirements of any lower class. Products classified in A1 and A2 classes are non-combustible (for example cement, concrete, glass, fiberglass, rock wool, ceramic etc.), materials certified from B to F are combustible in ascending order.

Flooring materials and linear pipe thermal insulation products are classified according to the same classes A1, A2, B, C, D, E and F followed by the abbreviation "fl" and "l", respectively.

All the products and materials classified as A2, B, C, D obtain an additional classification regarding the emission of smoke and the production of flaming droplets and/or particles:

s	Smoke emission during combustion	1	quantity/speed of emission absent or weak
		2	quantity/speed of emission of average intensity
		3	quantity/speed of emission of high intensity
d	Production of flaming droplets/particles during combustion	0	no dripping
		1	slow dripping
		2	high dripping

Classification of construction products and flooring materials according to standard EN 13501-1 is shown in Table 11.

Table 11
Classification according to European Standard EN 13501-1 [21]

Definition	Construction products			Flooring materials	
	A1			$A1_{fl}$	
Non-combustible materials	A2-s1,d0	A2-s1,d1	A2-s1,d2	$A2_{fl}$-s1	$A2_{fl}$-s2
	A2-s2,d0	A2-s2,d1	A2-s2,d2		
	A2-s3,d0	A2-s3,d1	A2-s3,d2		
Combustible materials-very limited contribution to fire	B-s1,d0	B-s1,d1	B-s1,d2	B_{fl}-s1	B_{fl}-s2
	B-s2,d0	B-s2,d1	B-s2,d2		
	B-s3,d0	B-s3,d1	B-s3,d2		
Combustible materials-limited contribution to fire	C-s1,d0	C-s1,d1	C-s1,d2	C_{fl}-s1	C_{fl}-s1
	C-s2,d0	C-s2,d1	C-s2,d2		
	C-s3,d0	C-s3,d1	C-s3,d2		
Combustible materials-medium contribution to fire	D-s1,d0	D-s1,d1	D-s1,d2	D_{fl}-s1	D_{fl}-s1
	D-s2,d0	D-s2,d1	D-s2,d2		
	D-s3,d0	D-s3,d1	D-s3,d2		
Combustible materials-highly contribution to fire	E	E-d2		E_{fl}	
Combustible materials-easily flammable	F			F_{fl}	

* For the E class is provided one single subclass d2.

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FIRE RESISTANCE OF STRUCTURES

Igor Džolev, Đorđe Lađinović

1. INTRODUCTION TO FIRE RESISTANCE OF STRUCTURES

1.1. Basic requirements

The general objectives of fire protection are to limit risks with respect to the individual and society, neighbouring property, and where required, environment or directly exposed property, in the case of fire [1]. According to the Construction Product Directive 89/106/EEC, the following basic requirements need to be fulfilled for the limitation of fire risks [2]:

- load bearing resistance needs to be provided for a specified period of time,
- generation and spread of fire and smoke need to be limited,
- spread of fire to neighbouring structures needs to be limited,
- safe evacuation of occupants need to be provided,
- safety of rescue teams needs to be taken into consideration.

These requirements impose additional considerations that need to be taken into account during the design phase, in form of passive and active fire protection measures, as to minimize the consequences in case the fire even takes place. Since, in general, such event cannot be predicted and eliminated as a threat (Figure 1), the engineering goal is to reduce the risks, by constantly upgrading the base of knowledge of the analysed phenomena and incorporating the solutions in engineering practice.

a) b) c)

Figure 1. Novi Sad (Serbia) Open University high-rise building fire on April 6, 2000: a) fire spread to the entire story of origin, b) vertical fire spread and c) current state of the building

Methods of fire resistance assessment, either by tests or calculation, can be divided in following categories [3]:

- standard fire tests,
- tabulated data (largely prescriptive but also increasingly based on calculations),
- simplified calculations, neglecting complex effects, such as thermal stresses),
- advanced calculations (largely performance based),
- full scale fire tests.

According to Eurocode, structures can be evaluated at three levels of increasing complexity:

- member analysis,
- substructure analysis,
- global structural analysis.

In addition to prescriptive and testing methods, current technical development allows the assessment of thermal and structural response to fire also by calculation. Experimental studies provide the most comprehensive knowledge on the behaviour of structures in fire. However, the costs of conducting such studies are substantial (experimental setup, equipment, specialized furnaces and instrumentation). Given the limitations in size of furnaces, and the costs of providing the equipment, as well as the large amounts of energy for each conducted test, the need for more sustainable approach has resulted in the development of calculation procedures to ensure an acceptable cost-benefit solution to engineering practice.

When assessing the fire resistance, irrespective of the method used, the first step is to model the real fire to a realistic and conservative fire scenario. In general, fire severity depends on a number of factors, including [3]:

- availability of combustible materials,
- ventilation conditions, in terms of oxygen delivery,
- physical characteristics of the space in which fire is initiated.

1.2. Fire action and fire models

Each real fire is unique, yet the same phases can be noticed during the course of fire: ignition, growth, flashover, fully developed fire stage, decay stage and extinguishment. For the purpose of structural fire analysis, depending on the assumptions and the level of complexity, fire models are divided into three categories:

- nominal fire curves,
- parametric fire curves,
- multi-zone models.

As the precise prediction of the fire start location, as well as the conditions in which fire will develop, are practically impossible to establish with certainty, in order to define a reference fire model to be used in the fire classification of structural elements, standard (nominal) fire curves are introduced. Most commonly used are ISO 834 fire curve [4], ASTM E119 [5], hydrocarbon and external fire curve [2] (Figure 2). Standard fire curves are derived from the data base of maximum temperatures registered in real cellulosic fires and represent the temperature evolution after flashover occurs. Mathematically, the curves represent the hot gas temperature evolution in a fully developed fire situation, in respect to time. The basic assumptions are that the temperature inside the fire compartment is considered independent of the compartment size and materialization, amount of combustible fuel present and the ventilation properties of the surrounding envelope. The temperature during fire is also considered independent of the spatial coordinates inside the fire compartment. The temperature-time functions are monotonically increasing, disregarding the cooling phase that follows after the fully developed fire phase. When using standard fires, fire resistance is measured in minutes as the time until a predefined failure criterion is met. Depending on the member function and topology, fire resistance is defined based on the following criteria:

- R - load bearing function (ability of a structure or a member to sustain specified actions during the relevant fire, according to defined criteria),
- E - integrity function (ability of a separating element, when exposed to fire on one side, to prevent the passage through it of flames and hot gases and to prevent the occurrence of flames on the unexposed side),
- I - insulation function (ability of a separating element when exposed to fire on one side, to restrict the temperature rise of the unexposed face below specified levels).

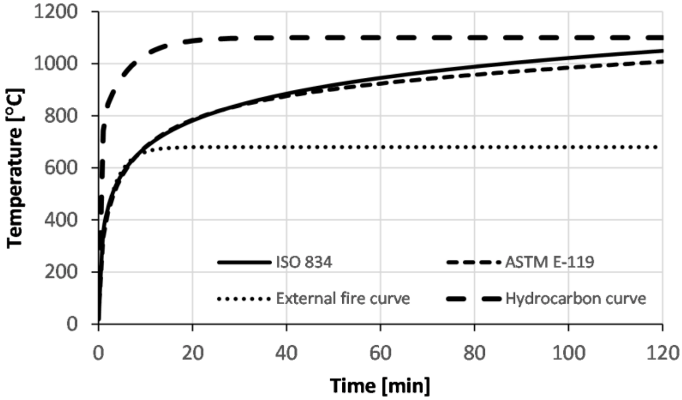


Figure 2. Standard temperature-time fire curves

If a standard fire exposure is adopted, the load bearing function is required for a certain period of time, while, for parametric fire exposure, the structure should be able to withstand the fire action for the whole duration of the fire, including the cooling phase. Load bearing and integrity function can only be assessed through thermal stress analysis and/or experimental tests. The insulation function, on the other hand, can be determined only by means of the heat transfer. Usually, the insulation criteria is assumed to be satisfied if the average temperature rise over the whole of the non-exposed surface is limited to 140°C and the maximum temperature rise at any point of that surface does not exceed 180°C. The insulation criteria should prevent spontaneous ignition of the fuel load outside the fire compartment, preventing the fire spread to neighbouring structures and compartments.

The member is then classified using the markings denoting the resistance criterion and the minimum duration of the standard fire (in minutes) until failure criterion is met (e.g. “REI 60” provides load bearing, integrity and insulation function of a member for at least 60 minutes of standard fire exposure).

Although the use of nominal fire curves provides comparable solution for the fire resistance classification of members, a large deviation of the temperature-time evolution in comparison with real fires can be observed depending on the size of compartment, amount of fire load available, etc., often providing conservative solution, but also in certain cases, a solution which is not on the safe side. For a more detailed assessment of a fire that could develop in a specific fire compartment, parametric fire curves could be used, taking into account real geometric and material properties of the compartment, as well as ventilation conditions. Parametric fire curves, unlike standard fire curves, also include a cooling (decay) phase of the fire, providing temperature-time evolution during the whole course of fire. In the design procedure, when using parametric fire curves, it is necessary to prove that the structure possesses an adequate fire resistance during the entire duration of the fire, including the cooling phase, as well as the phase after the fire is completely extinguished. The latter, depending on the primary structural material, can be crucial, since for materials with large thermal inertia, peak temperatures in members, due to transient heat transfer effects, may occur when the fire is completely put out. This could be very important for the fire fighters, rescue service and first responders entering the building immediately after the event. This type of design approach is specific for performance-based design (PBD), which is increasingly in use nowadays, since unique contemporary architecture, use of modern materials and bold design solutions often cannot be comprehended using prescriptive design procedures.

An accurate fire model is fundamental part of fire-structure modelling. Although accurate models are still not available for post-flashover fires in non-combustible compartments, extensive research is being conducted in the last years.

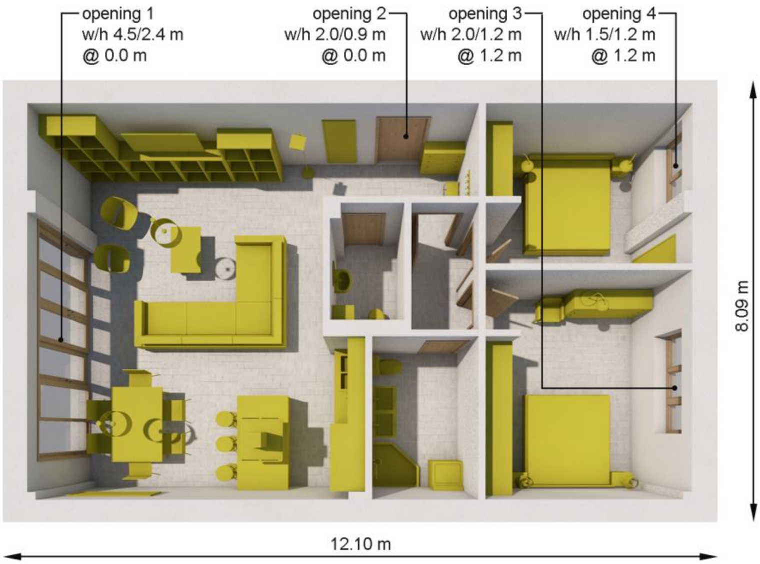
In case a more accurate assessment of temperature development within the fire sector is needed, zone models, based on mass and energy conservation laws can be applied. Due to the complexity of the numerical calculation, iterative procedure is needed, conditioning the use of these models to specialized computer software.

An arbitrary fire compartment can therefore be analysed using different fire models, depending on the analysis objective and the level of uncertainty in case a fire occurs. An example of a residential dwelling under consideration is presented in Figure 3. A three-room family apartment is considered as one fire compartment, where the geometry, layout, openings and the layers of enclosure are well defined (Table 1).

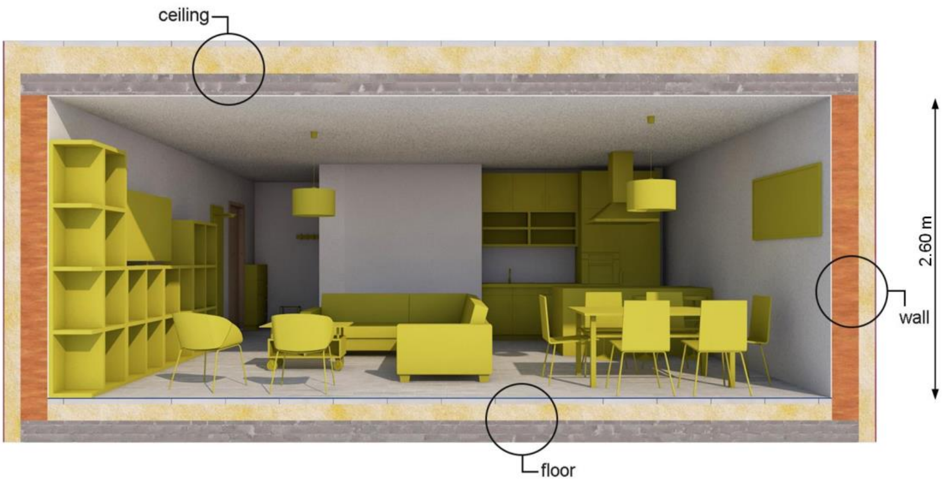
Table 1
Layers of the compartment enclosure

	Material	Thickness [cm]	Unit mass [kg/m^3]	Conductivity [W/mK]	Specific heat [J/kgK]
Floor	Ceramic tiles	1	2300	1.28	920
	Concrete screed	5	2200	1.40	1050
	Rock wool	15	60	0.037	1030
	Concrete	20	2300	1.60	1000
Ceiling	Mortar	1	1700	0.85	1050
	Concrete	20	2300	1.60	1000
	Rock wool	25	60	0.037	1030
	Concrete screed	5	2200	1.40	1050
Wall	Mortar	1	1700	0.85	1050
	Thermo-block	25	1400	0.61	920

Rock wool	15	60	0.037	1030
Mortar	1	1700	0.85	1050



a)



b)

Figure 3. Geometry, openings definition and boundary layers of analysed compartment

Temperature-time curves developed for this particular compartment are presented in Figure 4. As previously described, for the subsequent structural fire analysis, different fire curves can be utilized, from simple (ISO 834), parametric (defined according to Annex A of EN 1991-1-2) to more complex, zone model, which incorporates compartment physical properties. It is important to outline that standard and parametric fire curves are post-flashover fires, which do not account for the duration of the growth phase following the ignition (stage “I”). If a simple comparison of the developed temperatures is needed, the origin of standard and parametric fire should be translated to the time of flashover (stage “II”), determined based on the zone model, developed in the computer software OZone [6, 7]. Standard fire curve, besides stage “I” and “II”, also does not consider the decay phase of the fire (stage “IV”).

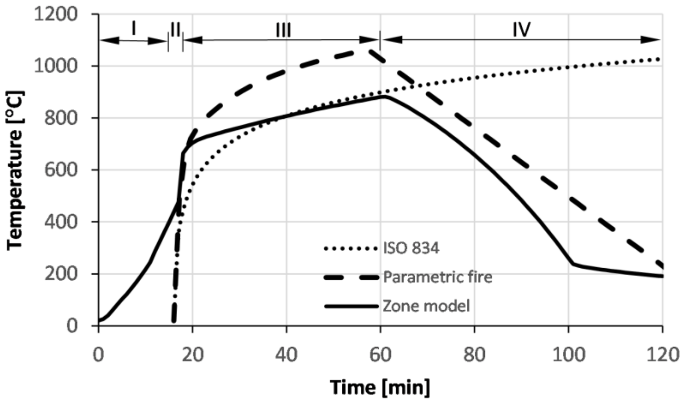


Figure 4. Temperature-time curves corresponding to analysed compartment

Once a temperature evolution of the hot gas in the compartment is determined, it can be used as an input to determine heat penetration inside structural members in time. The thermal analysis outcome should provide temperature profiles in a space and time manner, needed e.g. for the determination of the insulation function of a separating member (bearing or non-load bearing), or for the assessment of strength and stiffness degradation of the bearing members, if the goal is to determine their load bearing function. Depending on the analysis goal, different fire resistance criteria can be assessed and the member/substructure/global structure fire resistance can be determined.

2. STRUCTURAL FIRE ANALYSIS METHODOLOGY

A structural fire design analysis should take into account the following steps as relevant [2]:

- selection of the relevant design fire scenarios,
- determination of the corresponding design fires,
- calculation of temperature evolution within the structural members,
- calculation of the mechanical behaviour of the structure exposed to fire.

A design fire scenario is a qualitative description of the fire development over time based on a fire risk assessment, which identifies key events that determine a fire and differentiates it from other possible fires. Typically, the process of ignition and fire growth, the state of a fully developed fire, cooling, as well as the environment within the building and systems that can affect the course of fire are defined.

Modern structural design methods require the use of sophisticated computer modelling to predict the actions from applied loads and fire exposure, and to predict the capacity of structures and structural members to resist those actions [8]. The main components of such model are shown in Figure 5.

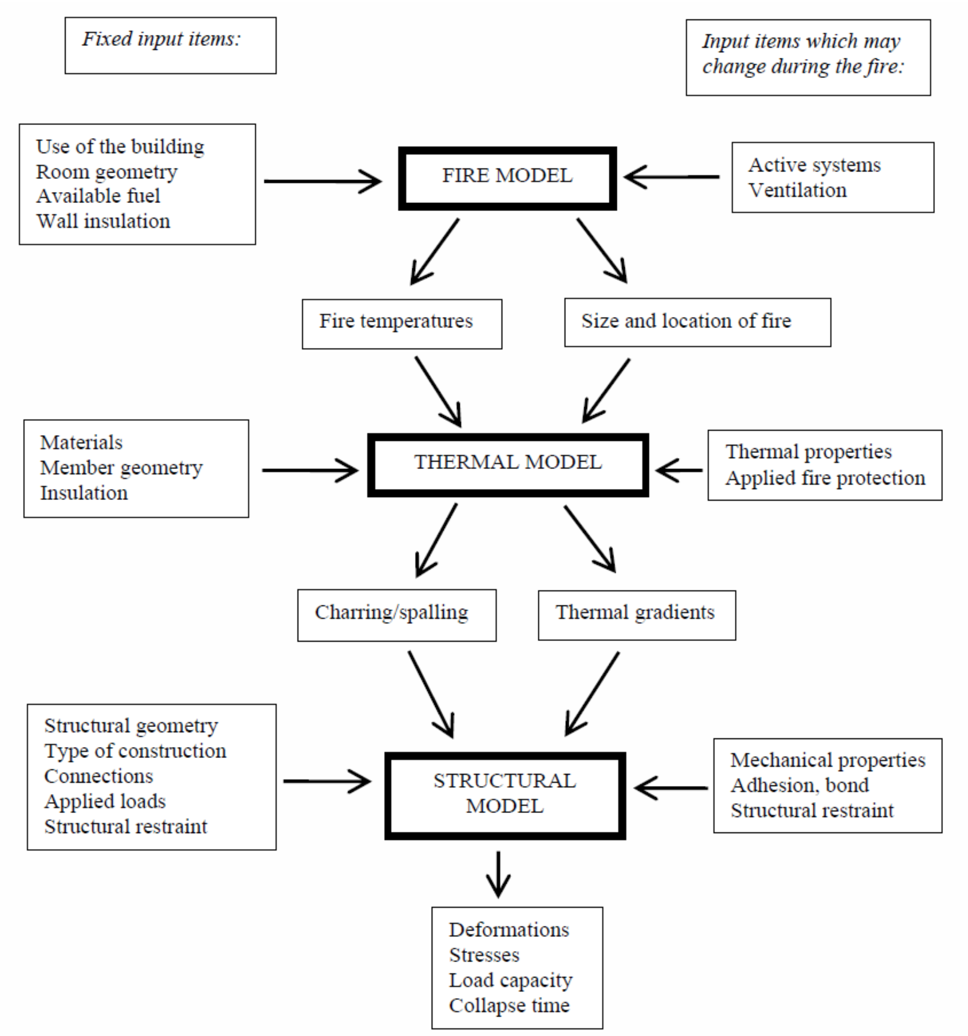


Figure 5. Flow chart for predicting structural fire performance

Advanced calculation of temperature evolution within the structural members is based on the transient heat transfer analysis, by means of conduction, convection and radiation. The governing differential equation for conductive heat transfer is:

$$\frac{\partial}{\partial x}\left(\lambda_x \frac{\partial T}{\partial x}\right)+\frac{\partial}{\partial y}\left(\lambda_y \frac{\partial T}{\partial y}\right)+\frac{\partial}{\partial z}\left(\lambda_z \frac{\partial T}{\partial z}\right)=\rho c\frac{\partial T}{\partial t} \qquad (1)$$

where:

- $\lambda_{x,y,z}$ is the thermal conductivity in all three directions (temperature dependent),
- ρ is the density of the material (temperature dependent),
- c is the specific heat (temperature dependent),
- T is the temperature,
- t is the time parameter.

The boundary conditions can be modelled in terms of both heat transfer mechanisms: convection and radiation.

The heat flux caused by convection is:

$$q_c = h_c \left(T_z - T_f \right) \tag{2}$$

where:

- h_c is the coefficient of convection (for wall in room at ambient temperature the recommended value is $h_c = 4\left[\mathrm{Wm^{-2}K^{-1}} \right]$, while in case of room fire, its recommended value is $h_c \geq 25\left[\mathrm{Wm^{-2}K^{-1}} \right]$),
- T_z is the temperature at the boundary of the element,
- T_f is the temperature of the fluid around the element.

The heat flux caused by radiation is:

$$q_r = V \varepsilon \sigma_c \left(T_{z,a}^4 - T_{f,a}^4 \right) = h_r \left(T_z - T_f \right) \tag{3}$$

$$h_r = V \varepsilon \sigma_c \left(T_{z,a}^2 + T_{f,a}^2 \right) \left(T_{z,a} + T_{f,a} \right) \tag{4}$$

where:

- h_r is the coefficient of radiation (temperature dependent),
- V is the radiation view factor (usually, $V = 1.0$),
- ε is the resultant coefficient of emission $\varepsilon = \varepsilon_f \varepsilon_z$, $\varepsilon_f = 1.0$ is the coefficient of emission for the surrounding fluid, ε_z is the coefficient of emission for the surface of the element, depending on the materialization (can be obtained from relevant Eurocode standards),
- $\sigma_c = 5.67 \cdot 10^{-8} \left[\mathrm{Wm^{-2}K^{-4}} \right]$ is the Stefan-Boltzmann constant,
- $T_{z,a}$ is the absolute temperature of the surface,
- $T_{f,a}$ is the absolute temperature of the fluid.

The solution to the differential equation is usually obtained using numerical procedures, e.g. finite element method (FEM).

Taking the radiation into account makes the problem nonlinear. This problem is solved by involving a new iterative procedure at every time step. The problem also becomes nonlinear when temperature dependent physical properties of the materials are assumed. In that case, the conductivity and capacity matrix are defined at the beginning of each time step based on the temperature from the previous step.

Calculation of the mechanical behaviour of the structure can be determined if the temperature fields are obtained during fire. Usually, first a heat transfer is calculated and the mechanical response in time is determined by taking into account the temperature distribution in members, for a constant gravitational load. This means that temperatures are calculated on undeformed geometry, which, in case of structural systems, is sufficiently accurate. Although fully coupled thermal-structural analysis would model the actual physical phenomenon more realistically, the calculation procedure would result in finding a solution to the coupled sets of equations at each time step of the analysis. This introduces additional degrees of freedom and becomes computationally more demanding. Since response accuracy is practically unaffected, the structural analysis is conducted after the temperature fields are determined.

The structural model should be based on fundamental physical behaviour. It should be derived from continuum mechanics, starting from linear elasticity and expanding to include plasticity and damage evolution, beyond the linear elastic formulation, since the structural response in fire is highly nonlinear. Nonlinearity is caused by the changes in material properties (both thermal and mechanical), as well as by the nonlinear temperature distribution in the element cross section. Also, for some types of structures, thermal expansion due to elevated temperatures can result in large deformations, which, for a realistic response assessment, requires taking into account geometric nonlinearity, as well. Given that analytical solutions are not developed, FEM is used, providing analysis of structures of arbitrary geometry, material and boundary conditions. In terms of accuracy, calculation models need to be verified (process of controlling the mathematical solution of the model) and validated (process of controlling the adequacy of the mathematical model representing real structural behaviour).

Besides advanced calculation models, which can predict the overall response of structures in fire with sufficient accuracy, but are overly complicated and impractical for everyday engineering practice, Eurocode standards provide simplified methods for fire resistance assessment of individual elements. Depending on the structural material, a list of standards to be used for structural fire design is presented in Table 2.

Table 2
List of Eurocode standards related to structural fire design

EN	Part	Title
EN 1990	n/a	Basis of structural design
EN 1991	1-2	Actions on structures - General actions - Actions on structures exposed to fire
EN 1992	1-2	Design of concrete structures - General rules - Structural fire design
EN 1993	1-2	Design of steel structures - General rules - Structural fire design
EN 1994	1-2	Design of composite steel and concrete structures - General rules - Structural fire design
EN 1995	1-2	Design of timber structures - General rules - Structural fire design
EN 1996	1-2	Design of masonry structures - General rules - Structural fire design
EN 1999	1-2	Design of aluminium structures - Structural fire design

3. INFLUENCE OF FIRES ON STRUCTURES

Fire action is considered as accidental. Indeed, the temperatures that are developing in the structural members are affecting the mechanical resistance of entire structure, which, if not properly considered, could result in structural failure and collapse of entire building.

High temperatures of the gases inside fire compartment, that are being developed when the fire load is ignited and burning cannot be suppressed, e.g. by the active fire protection measures (such as sprinklers), are heating the structural members by the heat transfer mechanism. Material properties that are affecting the temperature rise are thermal conductivity, specific heat and density. Thermal conductivity is a measure of the material ability to conduct heat. Heat transfer occurs at a lower rate in materials of low thermal conductivity than in materials of high thermal conductivity. For instance, metals typically have high thermal conductivity and are very efficient at conducting heat, resulting in faster penetration of heat and consequently, faster degradation of mechanical properties during fire. On the other hand, lower thermal conductivity, such as in concrete or timber structures, provide good insulating properties, which means that the temperature gradient is large and only the temperature of the outside layer is markedly increased, while the temperature on the internal parts of the element section remains comparatively low, retaining the load bearing capacity of a large portion of the section at close to ambient temperature level. Large temperature gradient induces high local stresses as a consequence of uneven thermal expansion of the part of the section. If those stresses exceed the strength of material, integrity of the section may be compromised. In reinforced concrete structures, high temperature gradient in the concrete cover (of the exposed member surfaces) could lead to chunks of concrete detaching from the member in a violent and explosive manner, phenomenon known as concrete spalling. One of the main parameters affecting concrete spalling in fire is the moisture content in members, since heating of the section would result in water vaporizing, increasing the pore pressure due to inability of free expansion, which induces additional pressures in the zones of interest. Other factors, such as the thickness of the concrete cover, size of the aggregate, rate of heating, porosity, permeability, as well as the applied stress level, could contribute to the evolution of spalling, which could be very hard to predict. If spalling occurs, reinforcement bars, otherwise protected by the concrete cover, will be directly exposed to burning flames. High thermal conductivity of steel would result in a faster heat transfer in the reinforcement, leading to a faster temperature rise and degradation of load bearing capacity, which could affect the overall resistance of the structure. It is essential, therefore, to assure that the probability of spalling occurring is minimized. For this purpose, the moisture content should be limited. Also, numerous efforts have been made in form of using additives in the concrete mix design, such as the polypropylene (PP) fibres in small amounts (ranging from 0 to 2% of the element volume), however, on the expense of concrete compressive strength degradation [9]. PP fibres, when uniformly distributed within concrete, play an active role in improving spalling resistance of concrete induced to elevated temperature. They have a relatively low melting point, after which they decompose (without producing noxious gases) and create space pockets, thus helping reduce the pressure in the pores during heating.

As opposed to concrete and wood, steel has relatively high thermal conductivity, resulting in fast heat transfer through the entire cross section and sudden turning point in terms of mechanical degradation of load bearing capacity. In order to postpone the temperature rise in steel during fire, assuring the desired fire resistance time, members are often protected by adding additional insulation materials (rock wool, plaster boards), or epoxy-based fire resistant coatings.

Specific heat of a material is the amount of heat to be supplied to a given mass of a material to produce a unit change of temperature. Materials with higher values of specific heat would therefore require larger amount of heat for the unit temperature change, resulting in material temperature change being delayed by a certain time phase compared to the external heat source temperature. In case of fire, this delay has beneficiary effect, postponing the temperature rise in structural elements and providing sufficient time for evacuation. However, at the later stages of fire, during the decay phase, when the gas temperature is getting lower, the temperature in elements might still continue to rise for some time, before starting to decline. This could be very dangerous for first responders and/or fire fighters entering the building after containing the fire.

Some materials, such as concrete and wood, exhibit density change at elevated temperatures, due to a loss of a free and chemically bounded water and/or chemical reactions that take place at higher temperatures. For steel, however, density remains constant in the entire range of expected temperatures in fire.

The essential requirement for structural fire safety may be observed by following various possibilities for fire safety strategies like conventional fire scenarios (nominal fires) or "natural" (parametric) fire scenarios, including passive and/or active fire protection measures.

Required functions and levels of performance can be specified either in terms of nominal (standard) fire resistance rating, generally given in national fire regulations or, where allowed by national fire regulations, by referring to fire safety engineering for assessing passive and active measures. Supplementary requirements concerning to the possible installation and maintenance of sprinkler systems; conditions on occupancy of building or fire compartment; the use of approved insulation and coating materials, including their maintenance are not given in this document, because they are subject to specification by the competent authority. Numerical values for partial factors and other reliability elements are given as recommended values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and quality management applies.

3.1. Concrete structures

Traditionally, concrete has been regarded as "fireproof" because of its incombustibility and relatively high thermal insulating properties. It is a versatile material and, if properly designed, can be inherently fire resistant. However, three main issues emerged from the concrete reaction to fire:

- deterioration of mechanical properties,
- damage caused by thermal deformations,
- spalling.

At the structural level, the development of fire engineering assessment methods came from the limitations inherent in the traditional prescriptive methods design. A set of conventions, rather than a rational approach with engineering tools, has its drawbacks, often being too conservative, but also not applicable for buildings of unique architectural and structural solution. In recent years, the whole package of conventions and requirements are re-examined in a holistic and scientific manner, advancing the field of structural fire engineering.

A full analytical procedure for structural fire design would take into account the behaviour of the structural system at elevated temperatures, the potential heat exposure and the beneficial effects of active and passive fire protection systems, together with the uncertainties associated with these three features and the importance of the structure (consequences of failure). At the present time it is possible to undertake a procedure for determining adequate performance which incorporates some, if not all, of these parameters and to demonstrate that the structure, or its components, will give adequate performance in a real building fire. However where the procedure is based on a nominal (standard) fire, the classification system, which calls for specific periods of fire resistance, takes into account (though not explicitly) the features and uncertainties described above. The prescriptive approach and the performance-based approach are identified. The prescriptive approach uses nominal fires to generate thermal actions. The performance-based approach, using fire safety engineering, refers to thermal actions based on physical and chemical parameters.

According to EN 1992-1-2, the design values of concrete mechanical properties $X_{\mathrm{d,fi}}$ (both strength and deformation) are defined as:

$$X_{\mathrm{d,fi}} = k_{\theta}\, X_{\mathrm{k}} / \gamma_{\mathrm{M.fi}} \tag{5}$$

where:

- X_{k} is the characteristic value of a strength or deformation property (generally f_{k} or E_{k}) for normal temperature design, according to EN 1992-1-1,
- k_{θ} is the reduction factor for a strength or deformation property ($X_{\mathrm{k},\theta} / X_{\mathrm{k}}$), dependent on the material temperature,
- $\gamma_{\mathrm{M.fi}}$ is the partial safety factor for the relevant material property, for the fire situation.

The design values of thermal material properties $X_{\mathrm{d,fi}}$ are defined as:

$$X_{\mathrm{d,fi}} = X_{\mathrm{k},\theta} / \gamma_{\mathrm{M.fi}} \tag{6}$$

if an increase of the property is favourable for safety, while:

$$X_{\mathrm{d,fi}} = \gamma_{\mathrm{M.fi}} X_{\mathrm{k},\theta} \tag{7}$$

if an increase of the property is unfavourable for safety, where:

- $X_{\mathrm{k},\theta}$ is the value of a material property in fire design, generally temperature dependent,
- $\gamma_{\mathrm{M.fi}}$ is the partial safety factor for the relevant material property, for the fire situation.

For reinforced concrete material, the recommended value of a partial safety factor is $\gamma_{\mathrm{M.fi}} = 1.0$, if otherwise specified in the National Annex.

Verification of fire resistance may be made in the time domain, in the strength domain or in the temperature domain. The required fire resistance time of the structure should be equal to or less than the available fire resistance time. Verification by strength is similar to the usual approach of structural design in ambient conditions. Critical temperature in the structure should be lower than the limiting temperature of the structure.

For the analysed structure, it should be proved that, for the relevant duration of fire exposure:

$$E_{\mathrm{d,fi}} \le R_{\mathrm{d,t,fi}} \tag{8}$$

where:

- $E_{\mathrm{d,fi}}$ is the design effect of actions for the fire situation, determined according to EN 1992-1-2, including effects of thermal expansions and deformations,
- $R_{\mathrm{d,t,fi}}$ is the corresponding design resistance in the fire situation.

As an alternative to design by calculation, fire design may be based on the results of fire tests, or on fire tests in combination with calculations. The latter is economically more feasible, since fire tests require specialized furnaces and equipment in order to prescribe the special thermal exposure conditions and to monitor the temperatures and deformations of structures at specific locations.

The effects of actions are the same as at ambient temperature. These actions are applied as constant values throughout the design fire exposure. As a substantial simplification, the effects of actions may be deduced from those determined in normal temperature design using the following expression:

$$E_{\mathrm{d,fi}} = \eta_{\mathrm{fi}} E_{\mathrm{d}} \tag{9}$$

where:

- E_{d} is the design value of the corresponding force or moment for normal temperature design, for a fundamental combination of actions, according to EN 1990,
- η_{fi} is the reduction factor for the design load level for the fire situation.

If the load combination for the fire design is:

$$G_{\mathrm{k}} + \psi_{\mathrm{fi}} Q_{\mathrm{k,1}} \tag{10}$$

and the load combination for normal temperature design is:

$$\gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}} \tag{11}$$

the reduction factor η_{fi} can be taken as:

$$\eta_{\mathrm{fi}} = \frac{G_{\mathrm{k}} + \psi_{\mathrm{fi}} Q_{\mathrm{k,1}}}{\gamma_{\mathrm{G}} G_{\mathrm{k}} + \gamma_{\mathrm{Q,1}} Q_{\mathrm{k,1}}} \tag{12}$$

where:

- $Q_{\mathrm{k,1}}$ is the principal variable load,
- G_{k} is the characteristic value of a permanent action,
- γ_{G} is the partial factor for a permanent action for normal temperature design,
- $\gamma_{\mathrm{Q,1}}$ is the partial factor for leading variable action for normal temperature design,
- ψ_{fi} is the combination factor for frequent or quasi-permanent values given either by $\psi_{1,1}$ or $\psi_{2,1}$.

As a further simplification, recommended value of $\eta_{\mathrm{fi}} = 0.7$ may be used for concrete structures.

The temperature profiles in concrete elements during fire are complex and non-uniform. Due to nonlinear temperature-dependency of physical and thermal properties of concrete, heat transfer analysis is usually calculated using numerical procedures. For structural elements exposed to nominal standard fire, Annex A of EN 1992-1-2 provides useful temperature profiles at different times of fire, for some standard dimensions of elements, such as for a slab exposed to fire from one side and some rectangular and circle cross-section elements exposed to fire from all sides. Annex B provides two simplified calculation methods for concrete elements: the 500°C isotherm method and the zone method.

The 500°C isotherm method is based on the hypothesis that concrete at a temperature of more than 500°C is neglected in the load bearing capacity calculation, while below 500°C, it is assumed that full strength and capacity is retained, as at ambient temperature. This method is applicable to reinforced and prestressed concrete cross sections with respect to axial force and bending moment. The calculation procedure may be carried out as follows:

- First, a 500°C isotherm needs to be determined (either using Annex A of EN 1992-1-2 or by means of transient thermal finite element analysis), for the specified fire exposure, standard or parametric fire (some restrictions apply),
- Based on the isotherm position, a new effective width and height of the section is defined, excluding the damaged concrete, whose thickness is made equal to the average depth of the 500°C isotherm,
- The temperature of reinforcing bars in the tension and compression zones needs to be determined. The temperature of the individual reinforcing bar can be evaluated from the temperature profiles in Annex A and is taken as the temperature in the centre of the bar. Some of the reinforcing bars may fall outside the reduced cross-section. Despite this, they may be included in the calculation of the ultimate load bearing capacity of the fire exposed cross section,
- Based on the temperatures in reinforcement bars, reduced strength of steel is determined according to Figure 6,
- For a reduced cross section dimensions and reinforcement with reduced strength, conventional calculation method is applied for the determination of the ultimate load bearing capacity of the element,
- Comparison of the ultimate load bearing capacity and the design load effect needs to be made with the required resistance.

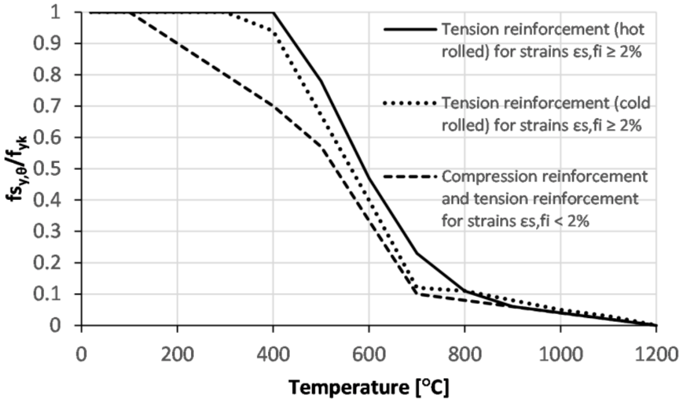


Figure 6. Class N reinforcement steel strength reduction at elevated temperatures

The 500°C isotherm method hypothesis is a relatively crude approximation of the concrete mechanical properties degradation, assuming discrete reduction of strength from 0 to 100% at a threshold temperature of 500°C. The rate of strength degradation with temperature is much more complex, affecting the accuracy of the proposed method. If a more accurate model is needed, at the expense of more labour, the zone method can be used, which can only be applied for the standard temperature-time curve.

The zone method is based on subdividing the cross section into several zones of equal thickness, where the mean temperature and the corresponding mean compressive strength and modulus of elasticity of each zone is assessed. The fire damaged cross section is represented by a reduced cross section ignoring a damaged zone of thickness a_z at the fire exposed sides (Figure 7).

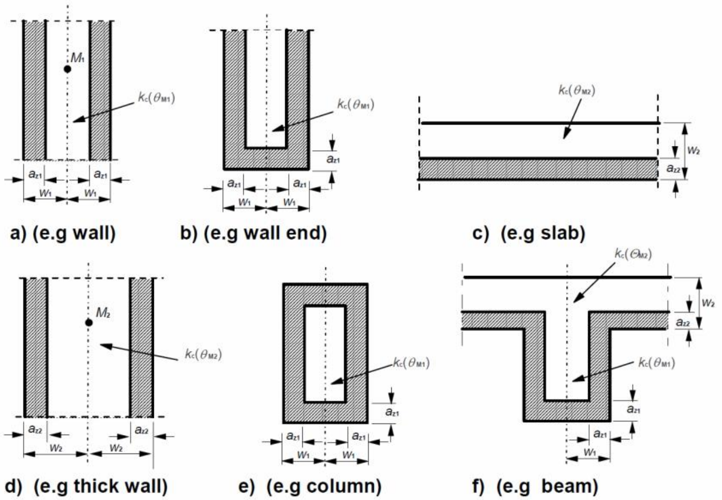


Figure 7. Reduction of strength and cross-section for sections exposed to fire

Reference is made to an equivalent wall. The point M is an arbitrary point on the centreline of the equivalent wall used to determine the reduced compressive strength for the whole of the reduced cross section. When two opposite sides are exposed to fire the width is assumed to be $2w$. For a rectangular cross section exposed to fire on one side only, the width is assumed to be w. This is represented by a wall width equal to $2w$. The flange is related to the first equivalent wall, and the web is related to the second equivalent wall. For the bottom and ends of rectangular members exposed to fire, where the width is less than the height, the value of a_z is assumed to be the same as the calculated values for the sides. The reduction of the cross section is based on a damaged zone of thickness a_z at the fire exposed surfaces. The damaged zone, a_z, is estimated for an equivalent wall exposed on both sides. The half thickness of the wall is divided into n parallel zones of equal thickness, where $n \geq 3$ (Figure 8). The temperature is calculated for the middle of each zone. The corresponding reduction factor for compressive strength, $k_c(\theta_i)$ is determined from the Figure 9.

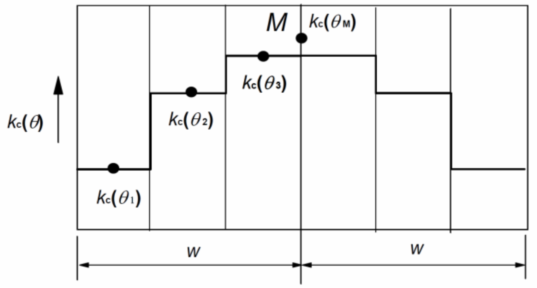
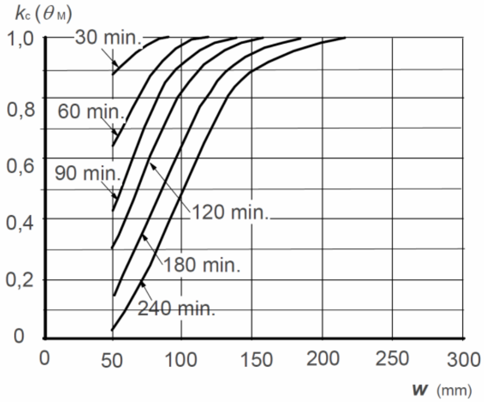


Figure 8. Division of a wall exposed to fire from both sides, into zones for use in calculation of strength reduction and a_z values



w is assessed as:

- The thickness of a slab
- The thickness of a one sided exposed wall or column
- Half the thickness of the web of a beam
- Half the thickness of a two sided exposed wall or column
- Half the smallest dimension of a four sided exposed column

Figure 9. Reduction of compression strength for a reduced cross-section using siliceous aggregate concrete

The mean reduction coefficient for a particular section may be calculated approximately by following expression:

$$k_{\mathrm{c,m}} = \frac{1-\dfrac{0.2}{n}}{n}\sum_{i=1}^{n} k_{\mathrm{c}}\left(\theta_{\mathrm{i}}\right) \tag{13}$$

where:

- n is the number of parallel zones in width w,
- w is half the total width,
- i is the zone number.

The width of the damaged zone for beams, slabs or members in plane shear may be calculated as:

$$a_{\mathrm{z}} = w\left[1-\frac{k_{\mathrm{c,m}}}{k_{\mathrm{c}}\left(\theta_{\mathrm{M}}\right)}\right] \tag{14}$$

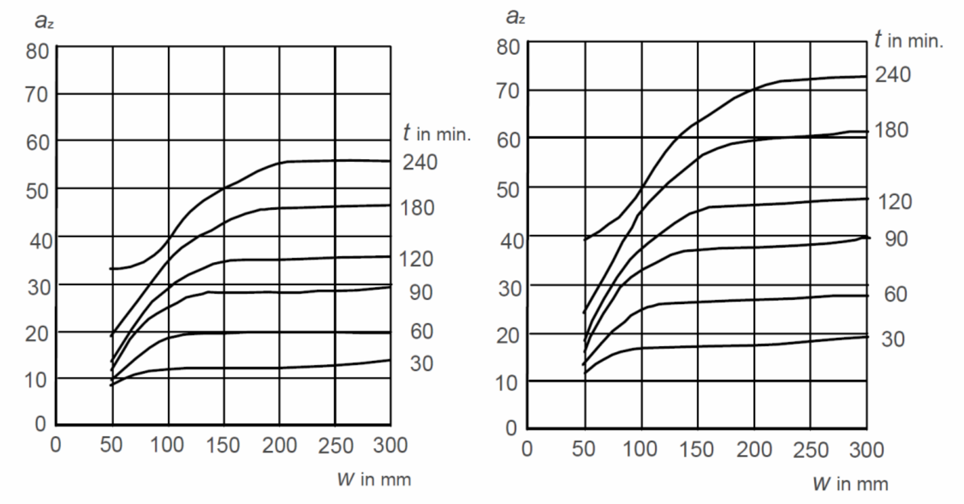
where:

- $k_{\mathrm{c}}\left(\theta_{\mathrm{M}}\right)$ is the reduction coefficient for concrete at point M.

For columns, walls and other structural elements where second order effects may be significant, the width of damaged zone may be calculated as:

$$a_z = w\left[1-\left(\frac{k_{c,m}}{k_c(\theta_M)}\right)^{1.3}\right] \tag{15}$$

Values for a_z can be obtained using the graphs in Figure 10. The values are calculated for siliceous aggregate concrete and are conservative for most other aggregates. When the reduced cross-section is found and the strength and modulus of elasticity are determined for the fire situation, the fire design follows the normal temperature design procedure by using $\gamma_{M.fi}$ values.



a) b)

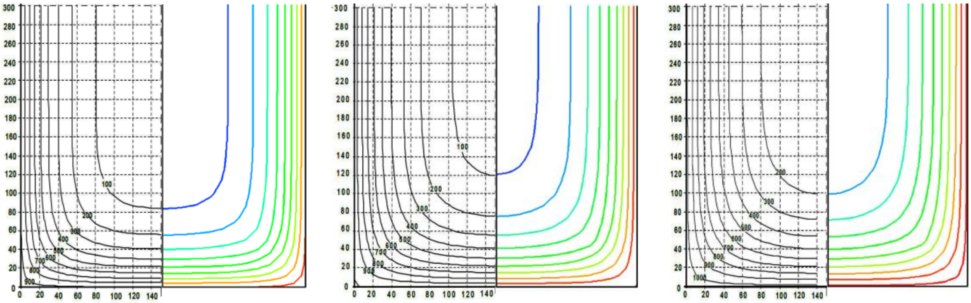
Figure 10. Reduction in cross section a_z of: a) a beam or slab, b) a column or wall

If a more detailed analysis is needed (e.g. for complex cross section forms or if simplified calculation method is simply not applicable), advanced calculation method can be used to verify the fire resistance of member/substructure/structure. Fire engineering calculation methods can be classified into three categories of increasing sophistication and complexity:

- simplified calculations based on limit state analysis (described above),
- thermo-mechanical finite element analysis,
- comprehensive thermo-hydro-mechanical finite element analysis.

When fire resistance is assessed by calculation, the first step would be to determine the thermal response in the form of time-dependent temperature distribution in the concrete structure. In integrated models the temperature distribution would be calculated along with the hydral and mechanical states for each time step. The thermal response provides the final answer only for the thermal insulation criterion but not for the load-bearing capacity or the complete answer for separating function (integrity).

Knowledge of the development of temperature distribution in concrete structures is the first key step in the understanding of the structure's behaviour in fire. Air temperatures in fires can exceed 900°C. However, the good insulating properties of concrete mean that the temperature gradient is large and only the temperature of the outside layer is markedly increased, while the temperature of the internal concrete remains comparatively low (Figure 11).

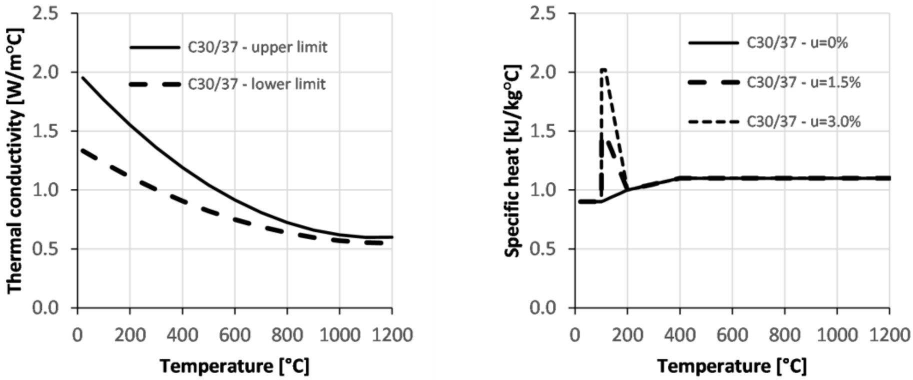


R60 R90 R120

Figure 11. Temperature profiles after 60, 90 and 120 minutes: EN 1992-1-2 (left) and ANSYS FEM thermal model (right) [10]

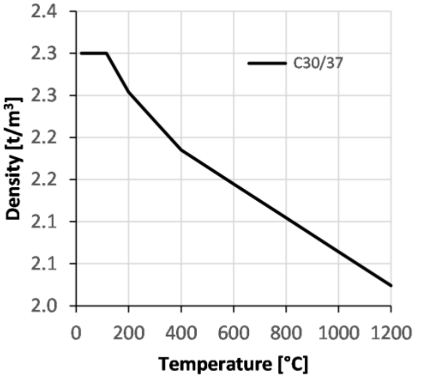
The thermal and mechanical analysis are normally interfaced and not integrated. The thermal analysis is carried out first and then fed into the mechanical analysis program to obtain the structural response. Moisture effects are usually not considered explicitly, but are accounted for, through modifications of the concrete material specific heat at elevated temperatures.

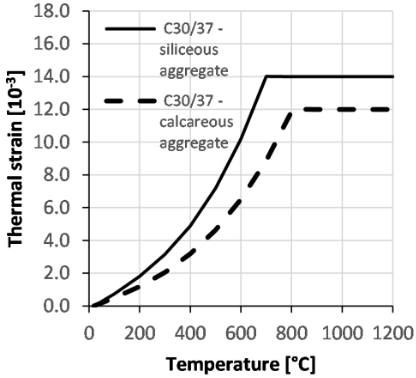
Nevertheless, the deformations of simple elements can be reasonably accurately predicted by such methods, providing that the load induced thermal strain (LITS) is incorporated into the model. The absence of moisture migration analysis means that evaporation plateau and explosive spalling cannot be predicted. EN 1992-1-2 provides temperature dependent properties for concrete that can be used if advanced calculation analysis is needed (Figure 12), where the stress-strain relations for concrete implicitly account for LITS effects.



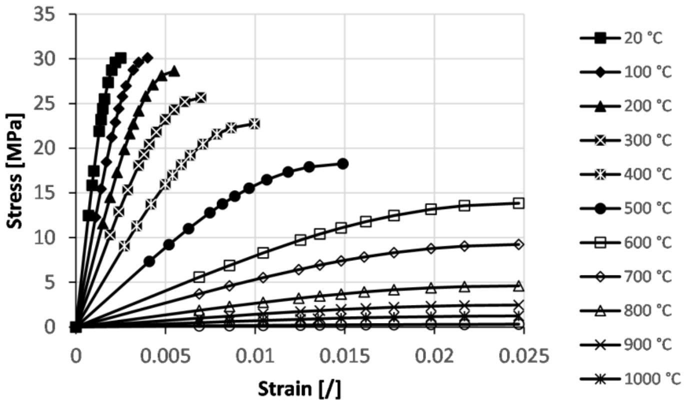
a) thermal conductivity b) specific heat

Figure 12. Temperature dependent thermal, physical and mechanical properties of concrete

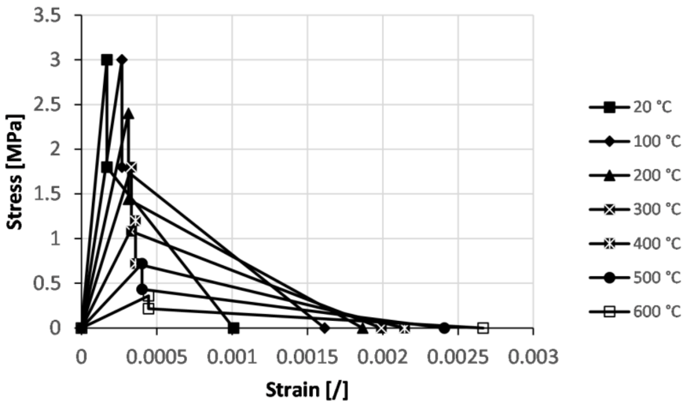




c) density d) thermal expansion



e) stress-strain relations in compression



f) stress-strain relations in tension

Figure 12. Temperature dependent thermal, physical and mechanical properties of concrete (Cont'd)

A comprehensive analysis would include thermal, hydral and mechanical analyses in a fully integrated model capable of predicting explosive spalling or the moisture state of the concrete, e.g. for nuclear reactors. In most practical cases, however, simpler methods are adequate for the analysis of structures in fire.

3.2. Steel structures

The past two decades have seen great advances in understanding the behaviour of steel in fire, and it can now justifiably be claimed that more is known about steel than any other framing material in fire. Steel is isotropic homogeneous material. Unlike concrete, which is composed of aggregate and cement paste and has considerably different behaviour in tension and compression, affected by various parameters (water to cement ratio, aggregate size, etc.), or wood, having different mechanical behaviour in directions parallel and perpendicular to the grains, steel micro- and macroscale properties are the same.

Fire resistance of structural steel elements is a function of the size of the section, its degree of exposure to the fire and the load that it carries. The strength of hot rolled structural steel decreases with temperature. Following an extensive series of standard fire tests, the strength reduction has been quantified. Recent research has also shown that the limiting (failure) temperature of a structural steel member is not fixed but varies according to two factors, the temperature profile and the load.

For small, fully loaded hot rolled sections, exposed on all four sides, the inherent fire resistance without added protection can be as little as 12 minutes. For very large, hot rolled sections, lightly loaded and with some partial protection from concrete floor slabs on the upper flange, this can be as high as 50 minutes. Where the heated perimeter is further reduced by the method of the construction (e.g. shallow floor systems), up to 60 minutes inherent fire resistance can be achieved. This is considerably less compared to reinforced concrete structures. Desired fire resistance of steel is achieved not on the material level, but with the application of passive (protection materials and coatings) and active protection measures ("sprinkler" system). Nevertheless, these measures may or may not be sufficient to contain the fire from spreading and developing to its full potential. If the fire is not suppressed during the growth phase, eventually, flashover will occur (if sufficient oxygen is employed and enough fuel load is present), which will result in steel members mechanical resistance being reduced, affecting the stability and load bearing of the structure.

For everyday engineering practice, simplified methods for analysing structural fire resistance have been developed, taking into account strength and stiffness degradation at elevated temperatures.

According to EN 1993-1-2 [11], the design values of thermal and mechanical material properties are defined as in EN 1992-1-2, i.e. expressions (5)-(12) also apply for steel material properties. As a simplification, recommended value of $\eta_{\mathrm{fi}} = 0.65$ may be used for steel structures, except for load categories for areas susceptible to accumulation of goods, including access areas, where the recommended value is $\eta_{\mathrm{fi}} = 0.7$.

For heating rates between 2 and 50°C/min (corresponding to most of the building fires), strength and deformation properties of steel at elevated temperatures should be obtained from the stress-strain relationships provided in EN 1993-1-2. These should be used to determine the resistances to tension, compression, moment and shear. The reduction factors for the stress-strain relationship are defined as follows:

$$k_{y,\theta} = f_{y,\theta} / f_y \tag{16}$$

$$k_{p,\theta} = f_{p,\theta} / f_y \tag{17}$$

$$k_{E,\theta} = E_{a,\theta} / E_a \tag{18}$$

where:

- $k_{y,\theta}$ corresponds to effective yield strengths, relative to yield strength at 20°C,
- $k_{p,\theta}$ corresponds to proportional limit, relative to yield strength at 20°C,
- $k_{E,\theta}$ corresponds to slope of linear elastic range, relative to slope at 20°C.

Temperature dependency of the reduction factors is presented in Figure 13. Although at temperatures below 400°C, proportional limit of linear elastic response, as well as the stiffness of the steel, are reduced as the temperature rises, the strength remains as for ambient temperature. However, after exceeding 400°C, the load bearing capacity is rapidly reduced, up to 77% in the temperature range 400-700°C. Therefore, in steel fire design, attempts should be made such that the temperature rise above 400°C is postponed for the required time of fire resistance, usually either by adding thermal insulation around the surfaces of the elements that could be exposed to fire, or by applying fire resistance coatings, thus, protecting the surfaces from direct exposure.

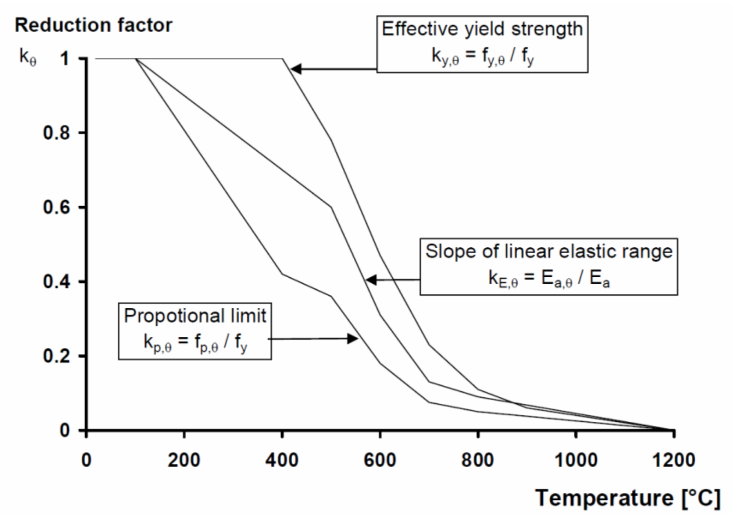


Figure 13. Reduction factors for the stress-strain relationship of carbon steel at elevated temperatures [EN 1993-1-2]

The design resistance $R_{d,t,fi}$ (eq. (8)) at time t shall be determined, usually in the hypothesis of a uniform temperature in the cross section, by modifying the design resistance for normal temperature design to EN 1993-1-1, to take account of the mechanical properties of steel at elevated temperatures. If a non-uniform temperature distribution is used, the design resistance for normal temperature design is modified on the base of this temperature distribution. Alternatively, by using a uniform temperature distribution, the verification may be carried out in the temperature domain.

Net section failure at fastener holes does not need to be considered, provided that there is a fastener in each hole, because the steel temperature is lower at connections due to the presence of additional material. The fire resistance of bolted or a welded connection may be assumed to be sufficient if the following is satisfied:

- The thermal resistance $(d_f/\lambda_f)_c$ of the connection's fire protection should be greater than the minimum value of thermal resistance $(d_f/\lambda_f)_m$ of fire protection applied to any of the jointed members,
- The utilization of the connection should be less than the maximum value of utilization of any of the connected members,
- The resistance of the connection at ambient temperature should satisfy the recommendations given in EN 1993-1-8 (Design of joints),

where:

- d_f is the thickness of the fire protection material ($d_f = 0$ for unprotected members),
- λ_f is the effective thermal conductivity of the fire protection material.

Classification of cross-sections should be made as for normal temperature design without considering any change by increasing temperature [11]:

- Class 1 cross sections are those which can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance,
- Class 2 cross sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling,
- Class 3 cross sections are those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic moment resistance,
- Class 4 cross sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross section.

The design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform temperature θ_a should be determined based on:

$$N_{fi,\theta,Rd} = k_{y,\theta} N_{Rd} \left[\gamma_{M,1} / \gamma_{M,fi} \right] \tag{19}$$

where:

- $k_{y,\theta}$ is the reduction factor for the yield strength of steel at temperature θ_a, reached at time t,
- N_{Rd} is the design resistance of the cross section for normal temperature design.

The design resistance $N_{fi,t,Rd}$ at time t of a tension member with non-uniform temperature distribution across the cross section may be determined from:

$$N_{fi,t,Rd} = \sum_{i=1}^{n} A_i k_{y,\theta,i} \; f_y / \gamma_{M,fi} \tag{20}$$

where:

- A_i is an elemental area of the cross section with a temperature θ_i,
- $k_{y,\theta,i}$ is the reduction factor for the yield strength of steel at temperature θ_i,
- θ_i is the temperature in the elemental area A_i.

The design resistance $N_{fi,t,Rd}$ at time t of a tension member with non-uniform temperature distribution may conservatively be taken as equal to the design resistance $N_{fi,\theta,Rd}$ of a tension member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

The design buckling resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a Class 1, Class 2 or Class 3 cross-section with a uniform temperature θ_a should be determined from:

$$N_{b,fi,t,Rd} = \chi_{fi} A k_{y,\theta} f_y / \gamma_{M,fi} \tag{21}$$

where:

- χ_{fi} is the reduction factor for flexural buckling in the fire design situation,
- $k_{y,\theta}$ is the reduction factor for the yield strength of steel at elevated temperature θ_a reached at time t.

The value of χ_{fi} should be taken as the lesser of the values $\chi_{y,fi}$ and $\chi_{z,fi}$ determined according to:

$$\chi_{fi} = \frac{1}{\varphi_\theta + \sqrt{\varphi_\theta^2 - \overline{\lambda}_\theta^2}} \tag{22}$$

with:

$$\varphi_\theta = \frac{1}{2}\left[1 + \alpha \overline{\lambda}_\theta + \overline{\lambda}_\theta^2\right] \tag{23}$$

and:

$$\alpha = 0.65\sqrt{235 / f_y} \tag{24}$$

The non-dimensional slenderness $\overline{\lambda}_\theta$ for the temperature θ_a is given by:

$$\overline{\lambda}_\theta = \overline{\lambda}\left[k_{y,\theta} / k_{E,\theta}\right]^{0.5} \tag{25}$$

The buckling length of a column for the fire design situation should generally be determined as for normal temperature design, except for a braced frame where additional considerations are made (see EN 1993-1-2).

The design resistance $N_{b,fi,t,Rd}$ at time t of a compression member with a non-uniform temperature distribution may be taken as equal to the design resistance $N_{b,fi,\theta,Rd}$ of a compression member with a uniform steel temperature θ_a equal to the maximum steel temperature $\theta_{a,max}$ reached at time t.

The design moment resistance $M_{\mathrm{fi},\theta,\mathrm{Rd}}$ of a Class 1 or Class 2 cross section with a uniform temperature θ_{a} should be determined from:

$$M_{\mathrm{fi},\theta,\mathrm{Rd}} = k_{\mathrm{y},\theta} M_{\mathrm{Rd}} \left[\gamma_{\mathrm{M,1}} / \gamma_{\mathrm{M,fi}} \right] \tag{26}$$

where:

- M_{Rd} is the plastic moment resistance of the gross cross section $M_{\mathrm{pl,Rd}}$ for normal temperature design, according to EN 1993-1-1 or the reduced moment resistance for normal temperature design, allowing for the effects of shear if necessary.

The design moment resistance $M_{\mathrm{fi,t,Rd}}$ at time t of a Class 1 or Class 2 cross section with a non-uniform temperature distribution across the cross section may be determined from:

$$M_{\mathrm{fi,t,Rd}} = \sum_{i=1}^{n} A_{\mathrm{i}} z_{\mathrm{i}} k_{\mathrm{y},\theta,\mathrm{i}} \; f_{\mathrm{y,i}} / \gamma_{\mathrm{M,fi}} \tag{27}$$

where:

- z_{i} is the distance from the plastic neutral axis to the centroid of the elemental area A_{i},
- $f_{\mathrm{y,i}}$ is the nominal yield strength f_{y} for the elemental area A_{i} taken as positive on the compression side of the plastic neutral axis and negative on the tension side.

Alternatively, the design moment resistance may also be determined from:

$$M_{\mathrm{fi,t,Rd}} = M_{\mathrm{fi},\theta,\mathrm{Rd}} / \kappa_1 \, \kappa_2 \tag{28}$$

where:

- $M_{\mathrm{fi},\theta,\mathrm{Rd}}$ is the design moment resistance of the cross section which is not thermally influenced by the supports,
- κ_1 is an adaptation factor for non-uniform temperature across the cross section,
- κ_2 is an adaptation factor for non-uniform temperature along the beam.

The design lateral torsional buckling resistance $M_{\mathrm{b,fi,t,Rd}}$ at time t of a laterally unrestrained beam with a Class 1 or Class 2 cross-section should be determined from:

$$M_{\mathrm{b,fi,t,Rd}} = \chi_{\mathrm{LT,fi}} W_{\mathrm{pl,y}} \, k_{\mathrm{y},\theta,\mathrm{com}} \, f_{\mathrm{y}} / \gamma_{\mathrm{M,fi}} \tag{29}$$

where:

- $\chi_{\mathrm{LT,fi}}$ is the reduction factor for lateral torsional buckling in the fire design situation,
- $k_{\mathrm{y},\theta,\mathrm{com}}$ is the reduction factor for the yield strength of steel at the maximum temperature in the compression flange $\theta_{\mathrm{a,com}}$ reached at time t.

The value of $\chi_{\mathrm{LT,fi}}$ should be determined according to:

$$\chi_{\mathrm{LT,fi}} = \frac{1}{\phi_{\mathrm{LT},\theta,\mathrm{com}} + \sqrt{\left[\phi_{\mathrm{LT},\theta,\mathrm{com}}\right]^2 - \left[\bar{\lambda}_{\mathrm{LT},\theta,\mathrm{com}}\right]^2}} \tag{30}$$

with:

$$\phi_{\mathrm{LT,\theta,com}} = \frac{1}{2}\left[1+\alpha\bar{\lambda}_{\mathrm{LT,\theta,com}} + \left(\bar{\lambda}_{\mathrm{LT,\theta,com}}\right)^2\right] \tag{31}$$

and:

$$\alpha = 0.65\sqrt{235/f_{\mathrm{y}}} \tag{32}$$

The non-dimensional slenderness $\bar{\lambda}_{\theta}$ for the temperature θ_{a} is given by:

$$\bar{\lambda}_{\mathrm{LT,\theta,com}} = \bar{\lambda}_{\mathrm{LT}}\left[k_{\mathrm{y,\theta,com}}/k_{\mathrm{E,\theta,com}}\right]^{0.5} \tag{33}$$

where:

- $k_{\mathrm{E,\theta,com}}$ is the reduction factor for the slope of the linear elastic range at the maximum steel temperature in the compression flange $\theta_{\mathrm{a,com}}$ reached at time t.

The design shear resistance $V_{\mathrm{fi,t,Rd}}$ at time t of a Class 1 or Class 2 cross section should be determined from:

$$V_{\mathrm{fi,t,Rd}} = k_{\mathrm{y,\theta,web}}V_{\mathrm{Rd}}\left[\gamma_{\mathrm{M,1}}/\gamma_{\mathrm{M,fi}}\right] \tag{34}$$

where:

- V_{Rd} is the shear resistance of the gross cross section for normal temperature design,
- θ_{web} is the average temperature in the web of the section,
- $k_{\mathrm{y,\theta,web}}$ is the reduction factor for the yield strength of steel at the steel temperature θ_{web}.

The value of the adaptation factors for non-uniform temperature distribution across a cross section should be taken as described in Table 3.

Table 3
Values of the adaptation factors for non-uniform temperature distribution across a cross section

description	κ_1 or κ_2
for a beam exposed on all four sides	$\kappa_1 = 1.0$
for an unprotected beam exposed on three sides, with a composite or concrete slab on side four	$\kappa_1 = 0.70$
for a protected beam exposed on three sides, with a composite or concrete slab on side four	$\kappa_1 = 0.85$
at the supports of a statically indeterminate beam	$\kappa_2 = 0.85$
in all other cases	$\kappa_2 = 1.0$

The design moment resistance $M_{\mathrm{fi,t,Rd}}$ at time t of a Class 3 cross section with a uniform temperature distribution may be determined from:

$$M_{\mathrm{fi,t,Rd}} = k_{\mathrm{y,\theta}}M_{\mathrm{Rd}}\left[\gamma_{\mathrm{M,1}}/\gamma_{\mathrm{M,fi}}\right] \tag{35}$$

where:

- M_{Rd} is the elastic moment resistance of the gross cross section $M_{\mathrm{el,Rd}}$ for normal temperature design or the reduced moment resistance allowing for the effects of shear if necessary.

The design moment resistance $M_{\mathrm{fi,t,Rd}}$ at time t of a Class 3 cross section with a non-uniform temperature distribution may be determined from:

$$M_{\mathrm{fi,t,Rd}} = k_{\mathrm{y,\theta,max}} M_{\mathrm{Rd}} \left[\gamma_{\mathrm{M,1}} / \gamma_{\mathrm{M,fi}}\right] / \left(\kappa_1 \kappa_2\right) \tag{36}$$

where:

- M_{Rd} is the elastic moment resistance of the gross cross section,
- $k_{\mathrm{y,\theta,max}}$ is the reduction factor for the yield strength of steel at the maximum steel temperature $\theta_{\mathrm{a,max}}$ reached at time t.

The design buckling resistance moment $M_{\mathrm{b,fi,t,Rd}}$ at time t of a laterally unrestrained beam with a Class 3 cross section should be determined from:

$$M_{\mathrm{b,fi,t,Rd}} = \chi_{\mathrm{LT,fi}} W_{\mathrm{el,y}}\, k_{\mathrm{y,\theta,com}}\, f_{\mathrm{y}} / \gamma_{\mathrm{M,fi}} \tag{37}$$

The design shear resistance $V_{\mathrm{fi,t,Rd}}$ at time t of a Class 3 cross section should be determined from:

$$V_{\mathrm{fi,t,Rd}} = k_{\mathrm{y,\theta,web}} V_{\mathrm{Rd}} \left[\gamma_{\mathrm{M,1}} / \gamma_{\mathrm{M,fi}}\right] \tag{38}$$

For members subjected to a combined bending and axial compression, the reader is referred to EN 1993-1-2.

For members with Class 4 cross sections other than tension members, it may be assumed that eq. (8) is satisfied if at time t, the steel temperature is not more than 350°C.

As an alternative, verification may also be carried out in the temperature domain. Except when considering deformation criteria or when stability has to be taken into account, the critical temperature $\theta_{\mathrm{a,cr}}$ of carbon steel at time t for a uniform temperature distribution in a member may be determined for any degree of utilization μ_0 at time $t = 0$ using:

$$\theta_{\mathrm{a,cr}} = 39.19 \ln\left[\frac{1}{0.9674 \mu_0^{3.833}} - 1\right] + 482 \tag{39}$$

For members with Class 1, Class 2 or Class 3 cross sections and for tension members, the degree of utilization at time $t = 0$ may be obtained from:

$$\mu_0 = E_{\mathrm{fi,d}} / R_{\mathrm{fi,d,0}} \tag{40}$$

where:

- $R_{\mathrm{fi,d,0}}$ is the value of $R_{\mathrm{fi,d,t}}$ for time $t = 0$.

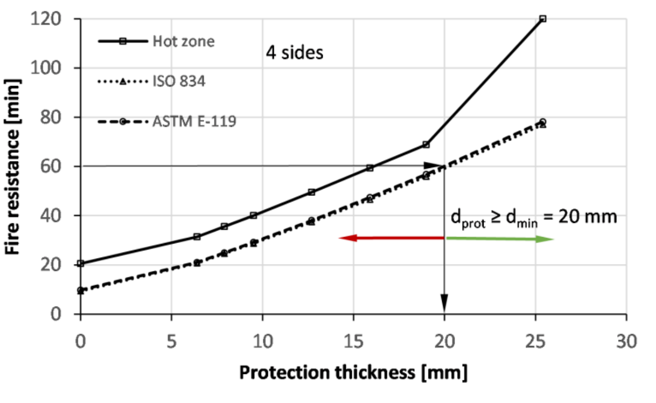
Specialized software OZone, used to calculate gas temperature evolution in the previously analysed compartment can also consider fire resistance of structural steel members, based on a standard fire exposure or hot zone temperature evolution determined from the zone model. In addition, the member can be exposed from three or all four sides and can be considered as unprotected or protected by the additional insulation material.

Considering a simply supported steel beam (Class 1) and taking into account permanent and variable action in the design, IPE 220 steel section has been adopted. Influence of the hollow encasement protection with gypsum boards on fire resistance is presented in Table 4. To meet the expected fire resistance criteria, thickness of the insulation layer in these cases can be obtained from the graph in Figure 14.

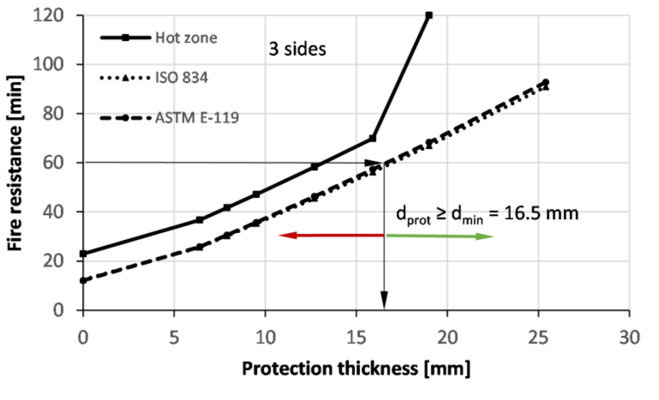
Table 4

Fire resistance of a simply supported steel beam and the influence of protection gypsum boards

IPE 220	Protection thickness [mm]	Exposed on Four Sides			Exposed on Three Sides		
Fire Resistance [min]		Zone model	ISO 834	ASTM E119	Zone model	ISO 834	ASTM E119
Unprotected	n/a	20.55	9.36	9.78	22.97	12.38	12.07
Protected (hollow encasement)	6.4	31.43	20.69	21.02	36.67	25.59	25.80
	7.9	35.54	24.57	24.91	41.70	30.26	30.56
	9.5	40.06	28.74	29.18	47.13	35.26	35.71
	12.7	49.52	37.4	38.07	58.25	45.50	46.31
	15.9	59.33	46.58	47.40	69.83	56.20	57.31
	19.0	68.82	55.92	56.77	> 120	66.97	68.29
	25.4	> 120	77.02	78.16	> 120	90.62	92.74



a) beam exposed on four sides

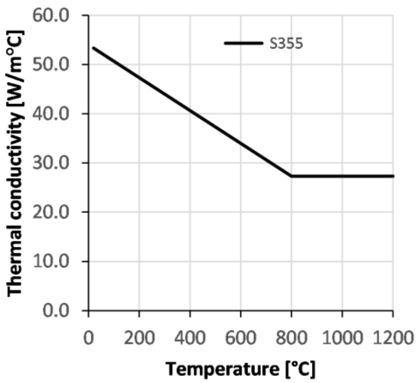


b) beam exposed on three sides

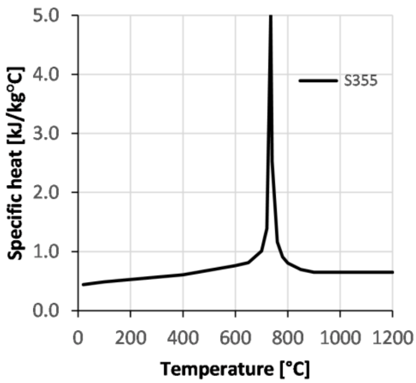
Figure 14. Fire resistance of the structural steel member depending on the protection layer thickness

As expected, fire resistance calculated for ISO 834 and ASTM E119 fire curve is practically the same. In case of the zone model, for this particular compartment, longer fire resistance is obtained. If, for example, R60 is needed (fire resistance of 60 minutes for standard fire exposure), the thickness of the protection layer needs to be at least 20 mm and 16.5 mm, in case where four and three sides of the member are exposed to fire, respectively.

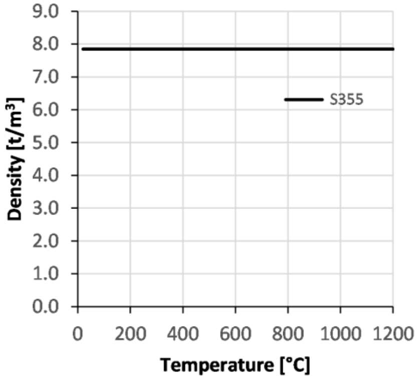
If the fire resistance is to be determined based on the advanced calculation method, steel thermal and mechanical properties need to be defined as temperature-dependent for the whole range of expected temperatures. EN 1993-1-2 provides information on the properties, presented in Figure 15. Unlike concrete and wood, which experience mass loss due to heating, steel density remains constant at elevated temperatures.



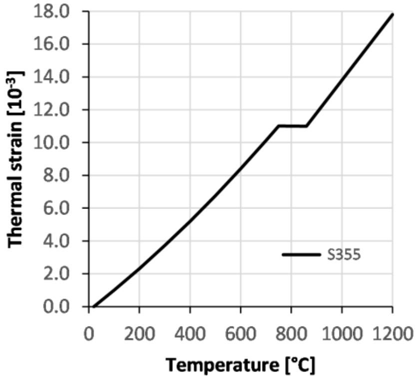
a) thermal conductivity



b) specific heat

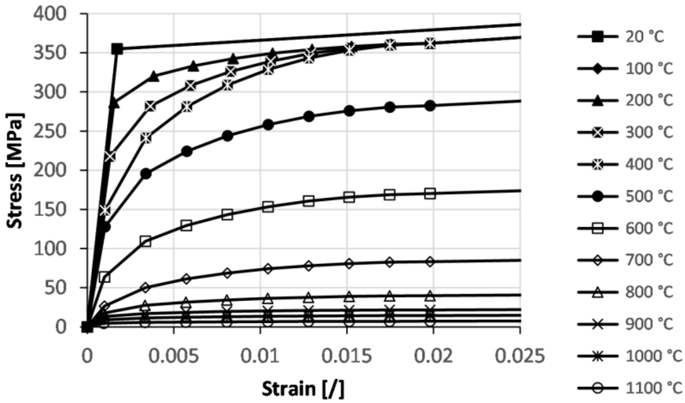


c) density



d) thermal expansion

Figure 15. Temperature dependent thermal, physical and mechanical properties of steel



e) stress-strain relations in tension and compression

Figure 15. Temperature dependent thermal, physical and mechanical properties of steel (Cont'd)

3.3. Timber structures

Wood may be considered as the oldest structural material. In the last two centuries of the modern age, other structural materials, such as concrete and steel, have become dominant, due to their high performance and durability. Production of concrete and steel, however, results in a large CO_2 emissions in the atmosphere, which is becoming an increasingly significant issue regarding the impact CO_2 has on climate conditions. In recent years, construction industry is trying to reduce the level of emissions, by promoting the use of wood as structural material, resulting in a rise of timber building projects around the globe. This has raised a number of potential problems that could appear in timber structures if fire occurs, since, unlike concrete and steel, wood is combustible material and will contribute to the overall fuel load during fire.

Automatic fire sprinkler systems are the most effective way of improving the fire safety of all buildings. They are especially recommended for use in tall timber buildings. In some cases, the encapsulation of timber elements is necessary, either complete or limited. Complete encapsulation provides sufficient thickness of gypsum plasterboard or other similar material to prevent charring of wood in a complete burnout, providing the same level of fire resistance as a totally non-combustible material. Limited encapsulation is a more economical solution which will prevent any involvement of the structural timber in the fire until well into the burning phase, but may not guarantee complete burnout with no onset of charring. Also, layered encapsulation is possible, referring to structural elements made up of layers of wood and non-combustible materials, to improve the appearance and the fire resistance.

The main risk for external fire spread is from big flames coming out of windows in a fully developed compartment fire and spreading upwards along the façade. There is no consensus or procedures on how to determine the risk for the external flames reaching two stories above the compartment fire. For timber structures, the main interest is to verify that wooden façades can be used in a fire safe way, also as façade claddings on e.g. concrete buildings [12].

Once a fire scenario and design fire are determined, structural fire response can be calculated by using simplified or advanced calculation methods. Simplified method currently widely used in fire design, is the reduced cross-section method and reduced properties method, which is proposed in EN 1995-1-2 [13].

Once a fire is developed, members which are directly exposed begin to heat up, as a consequence of the heat flux acting on a surface of a member. Heat begins penetrating the cross-section of a member by means of heat transfer. The heat penetration is relatively slow, given the low values of thermal conductivity of timber. When the temperatures reach values between 250 and 350°C (usually a 300°C threshold value is adopted), charring of timber occurs. Charring is a chemical process of incomplete combustion of certain solids when subjected to high heat. Heat distillation removes water vapour and volatile organic compounds from the matrix. The residual black carbon material is char, as distinguished from the lighter coloured ash. Although the charring layer does not contribute to the load bearing capacity, it protects the remaining part of the cross-section, acting as an insulation, by slowing the process of heat penetration. If the integrity of the layer is preserved, the core of the cross-section remains relatively cold, preserving the mechanical properties at the ambient temperature level. Besides the char layer, an additional transition layer between the charring and unaffected layer is formed, with degraded mechanical properties.

For a standard fire exposure, an effective cross-section for unprotected members should be calculated by reducing the initial cross-section by the effective charring depth d_{ef} (Figure 16), which can be determined by the following expression:

$$d_{\mathrm{ef}} = d_{\mathrm{char,n}} + k_0 d_0 \tag{41}$$

where:

- $d_0 = 7\ \mathrm{mm}$,
- k_0 is determined according to:

$$k_0 = \begin{cases} t/20 & \text{for } t < 20 \text{ min} \\ 1.0 & \text{for } t \geq 20 \text{ mm} \end{cases} \tag{42}$$

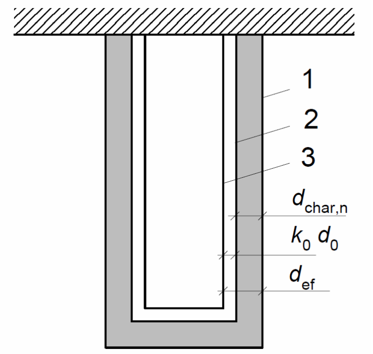
- $d_{\mathrm{char,n}}$ is determined according to:

$$d_{\text{char},0} = \beta_0 t \tag{43}$$

$$d_{\text{char},n} = \beta_n t \tag{44}$$

where:

- $d_{\text{char},0}$ is the design charring depth for one-dimensional charring,
- $d_{\text{char},n}$ is the notional design charring depth, which incorporates the effect of corner rounding,
- β_0 is the one-dimensional design charring rate under standard fire exposure (Figure 17),
- β_n is the notional design charring rate, the magnitude of which includes the effect of corner rounding and fissures (Figure 17),
- t is the time of fire exposure in minutes.



1 - Initial surface of member

2 - Border of residual cross-section

3 - Border of effective cross-section

Figure 16. Definition of residual cross-section and effective cross-section

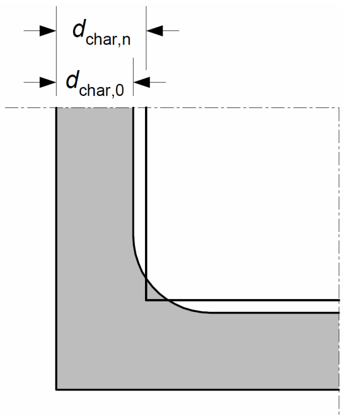


Figure 17. Charring depth $d_{\text{char},0}$ for one-dimensional charring and notional charring depth $d_{\text{char},n}$

The design strength and stiffness properties of the effective cross-section should be calculated with $k_{\text{mod,fi}} = 1.0$.

The one-dimensional design charring rate may be applied, provided that the increased charring rate near corners is taken into account, for cross-sections with an original width, b_{min}, where:

$$b_{\mathrm{min}} = \begin{cases} 2d_{\mathrm{char,0}} + 80 & \text{for } d_{\mathrm{char,0}} \geq 13 \text{ mm} \\ 8.15 d_{\mathrm{char,0}} & \text{for } d_{\mathrm{char,0}} < 13 \text{ mm} \end{cases} \tag{45}$$

When the smallest width of the cross-section is smaller than b_{min}, notional charring rates should be applied. For cross-sections calculated using one-dimensional design charring rates, the radius of the corner rounding should be taken equal to the charring depth $d_{\mathrm{char,0}}$.

For surfaces of timber, unprotected throughout the time of exposure, design charring rates are given in Table 5. Design charring rates for solid hardwoods, except beech, with characteristic densities between 290 and 450 kg/m^3, can be determined based on linear interpolation between the values given in Table 5. Charring rates of beech should be taken as given for solid softwood. Design charring rates for wood-based panels apply to a characteristic density of 450 kg/m^3 and a panel thickness of 20 mm. For other characteristic densities and panel thicknesses smaller than 20 mm, charring rate should be calculated as:

$$\beta_{0,\rho,\mathrm{t}} = \beta_0 k_\rho k_{\mathrm{h}} \tag{46}$$

with:

$$k_\rho = \sqrt{\frac{450}{\rho_k}} \tag{47}$$

$$k_h = \sqrt{\frac{20}{h_p}} \tag{48}$$

where:

- ρ_{k} is the characteristic density [kg/m^3],
- h_{p} is the panel thickness [mm].

Table 5
Design charring rates β_0 and β_{n} of timber

	β_0 [mm/min]	β_{n} [mm/min]
a) Softwood and beech		
Glued laminated timber with a characteristic density of ≥ 290 kg/m^3	0.65	0.7
Solid timber with a characteristic density of ≥ 290 kg/m^3	0.65	0.8
b) Hardwood		
Solid or glued laminated hardwood with a characteristic density of 290 kg/m^3	0.65	0.7
Solid or glued laminated hardwood with a characteristic density of ≥ 450 kg/m^3	0.50	0.55
c) LVL (laminated veneer lumber) With a characteristic density of ≥ 480 kg/m^3	0,65	0.7
d) Panels (characteristic density of 450 kg/m^3)		
Wood panelling	0.90	n/a
Plywood	1.00	
Wood-based panels other than plywood	0.90	

For protected surfaces, additional information can be found in EN 1995-1-2.

Reduced properties method can be applied for rectangular cross-sections of softwood exposed to fire on three or four sides and round cross-sections exposed along the perimeter.

For verification of mechanical resistance, the design values of strength and stiffness properties shall be determined from:

$$f_{\mathrm{d,fi}} = k_{\mathrm{mod,fi}} \frac{f_{20}}{\gamma_{\mathrm{M},fi}} \tag{49}$$

$$S_{\mathrm{d},fi} = k_{\mathrm{mod,fi}} \frac{S_{20}}{\gamma_{\mathrm{M},fi}} \tag{50}$$

where:

- $f_{\mathrm{d,fi}}$ is the design strength in fire,
- $S_{\mathrm{d,fi}}$ is the design stiffness property (modulus of elasticity $E_{\mathrm{d,fi}}$ or shear modulus $G_{\mathrm{d,fi}}$) in fire,
- f_{20} is the 20% fractile of a strength property at normal temperature,
- S_{20} is the 20% fractile of a stiffness property (modulus of elasticity or shear modulus) at normal temperature,
- $k_{\mathrm{mod,fi}}$ is the modification factor for fire,
- $\gamma_{\mathrm{M,fi}}$ is the partial safety factor for timber in fire.

The recommended partial safety factor for material properties in fire is $\gamma_{\mathrm{M,fi}} = 1.0$. The modification factor takes into account the reduction in strength and stiffness properties at elevated temperatures and replaces the modification factor for normal temperature design.

For $t \geq 20$ minutes, modification factor for fire $k_{\mathrm{mod,fi}}$ should be taken as:

- for bending strength:

$$k_{\mathrm{mod,fi}} = 1.0 - \frac{1}{200} \frac{p}{A_r} \tag{51}$$

- for compressive strength:

$$k_{\mathrm{mod,fi}} = 1.0 - \frac{1}{125} \frac{p}{A_r} \tag{52}$$

- for tensile strength and modulus of elasticity:

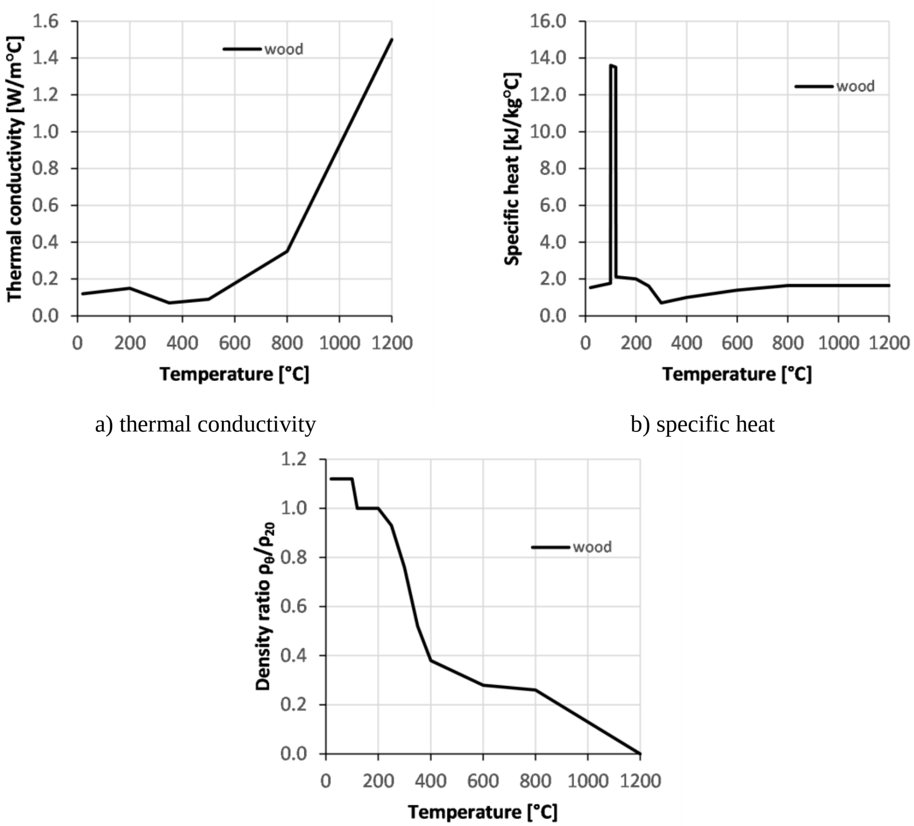
$$k_{\mathrm{mod,fi}} = 1.0 - \frac{1}{330} \frac{p}{A_r} \tag{53}$$

where:

- p is the perimeter of the fire exposed residual cross-section [m],
- A_{r} is the area of the residual cross-section [m^2].

For $t = 0$ minutes, the modification factor for fire should be taken as $k_{\mathrm{mod,fi}} = 1.0$ and for $0 \le t \le 20$ minutes, linear interpolation should be used.

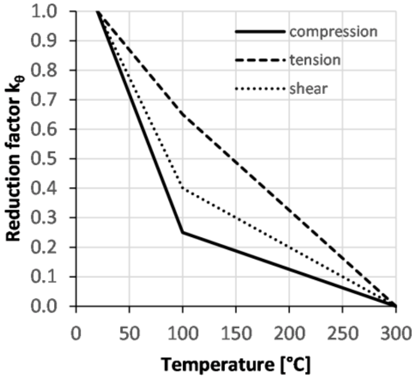
If a more detailed analysis is to be performed, an advanced calculation method can be used, either for individual members, parts of a structure or entire structures. Advanced methods may be applied for the determination of the charring depth, the development and distribution of the temperature within structural members and the evaluation of structural behaviour. The thermal response model should take into account variation of thermal properties of the material with temperature, while the structural response should take into account the changes of mechanical properties with temperature, as well as with moisture. Thermal and mechanical temperature dependent properties, as proposed in EN 1995-1-2, are presented in Figure 18.



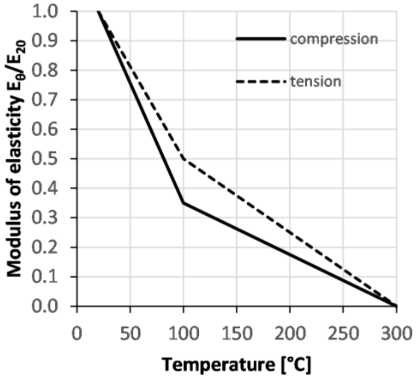
a) thermal conductivity

b) specific heat

c) density



d) reduction factor for strength parallel to grain of softwood



e) temperature effect on modulus of elasticity parallel to grain of softwood

Figure 18. Temperature dependent thermal, physical and mechanical properties of wood

In recent years, advancements are made in numerical modelling of structural response in fire, taking advantage of the computational resources that are constantly developing. Structural materials, such as concrete, steel and wood are being used simultaneously and are combined to comprehend for the strengths of each particular material. Contemporary structural systems, e.g. timber-concrete composite slabs, consist of a concrete slab (predominantly loaded in compression), supported by a timber beam (having higher strength in tension than concrete and being predominantly loaded in tension), with a connection between the two being in form of steel screws or plates (shear connection between the slab and the beam). Complex material behaviour and realistic modelling of the structural response is further complicated with the introduction of elevated temperatures, posing a challenge for engineers and researchers in the field. Further information on advanced numerical modelling of such systems can be found in [14].

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Section II

Assessment and Repair of Structures

FIRE DAMAGES OF REINFORCED CONCRETE STRUCTURES AND REPAIR POSSIBILITIES

Mirjana Malešev, Vlastimir Radonjanin

1. INTRODUCTION

Properly designed and successful execution of repair work of reinforced concrete (RC) structure damaged in fire can only be provided if a detailed in-situ and laboratory investigation and correct assessment of residual structural capacity have been made. Recommendations available from various sources (books, codes, articles etc.) could help to choose appropriate solution from a wide range of available repair methods and repair materials, but in practice every single fire damaged structure is unique [20]. For realistic assessment of the structure after a fire it is necessary to know behaviour of concrete and reinforcing steel at high temperature, to be able to recognize the type and degree of damage due to the fire and to separate them from similar damages that result from other causes. Reinforced concrete is considered a material that shows an acceptable resistance to high temperatures, which allows using concrete elements without the need of any additional protection. The main reason for this statement are the following properties of the concrete: incombustibility, small thermal conductivity, small strains at rising temperatures and therefore concrete core remains intact inside the section of element and continues transmit load. On the other hand, reinforcement is sensitive to high temperatures and needs to be protected. In RC structures concrete cover plays that role. The relatively low thermal conductivity of concrete leads to a slow propagation of chemical transformations of the components of concrete, which also need time for fully developing conversions at each specific temperature. On the other hand, low thermal conductivity of concrete causes strong thermal gradient that induce internal stresses in concrete mass and development of inner cracks [1]. However, long period of exposition of reinforced concrete to high temperatures introduce physical-chemical changes in its properties that lead to mechanical strength decay which produces losses in the bearing capacity and safety of the structure.

2. DAMAGE MECHANISMS OF CONCRETE UNDER FIRE

Concrete is a composite material that consists mainly of mineral aggregates bound by a matrix of hydrated cement paste. The matrix is highly porous and contains a relatively large amount of free water. When subjected to heat, concrete responds not just in instantaneous physical changes, such as expansion, but by undergoing various chemical changes. This response is especially complex due to the non-uniformity of the material. Concrete contains both cement and aggregate elements, and these may react to heating in a variety of ways [7]. The main changes occur primarily in the hardened cement paste. With the increase of temperature in concrete to 100^0C free water from the capillary pore system of hardened cement paste will be evaporated. In the range of 100^0C - 400^0C the cement paste loses physically bond water, while at temperatures above 400^0C chemically bound water will be lost. The following chemical transformations can be observed by increase of temperature: the decomposition of ettringite between 50^0C and 110^0C, endothermic dehydration of $Ca(OH)_2$ at the temperatures 450^0C-550^0C and dehydration of calcium-silicate-hydrates at the temperature of 700^0C [2]. The loss of pore water and chemical transformations are accompanied by shrinkage of cement stone. On the other hand, due to rising temperatures, coarse aggregate increases its volume and disruption of adhesion between the cement paste and coarse aggregate appeared. In the case of reinforced concrete, the same mechanism leads to impaired adhesion between the reinforcement and concrete [20].

Aggregate normally occupy 65 to 75% of the concrete volume, and that is why the behavior of concrete at elevated temperature is strongly influenced by the aggregate type. Commonly used aggregate materials are thermally stable up to 300^0C–350^0C. Aggregate used in concrete can be classified into three types: carbonate, siliceous and lightweight aggregate (LWA). Carbonate aggregates include limestone and dolomite. Siliceous aggregate include materials consisting of silica and include granite and sandstone. LWA are usually manufactured by heating shale, slate, or clay. Compressive strength of concrete containing siliceous aggregate begins to drop off at about 400^0C and is reduced to about 55% at 650^0C because of change of crystal structure of quartz α formation → β formation [2]. Concrete containing LWA and carbonate aggregates retain most of their compressive strength up to about 650^0C. Lightweight concrete has better insulating properties, and transmits heat at a slower rate than normal weight concrete with the same thickness, and therefore generally provides increased fire resistance. The modulus of elasticity for concretes manufactured of all three types of aggregates is reduced with the increase in temperature. Also, at high temperatures, creep and relaxation of concrete increase significantly. The colour of concrete generally changes at increasing temperature from normal to pink or red (300-600^0C), whitish grey (600-900^0C) and buff (900-1000^0C). If the concrete temperature exceeds 1300^0C, the softening and melting of surface layer will be occur [20]. Described physical and chemical changes in concrete will have the effect on reduction of the compressive strength of the material. Generally, concrete will maintain its compressive strength until a critical temperature is reached, above which point it will rapidly drop off. This generally occurs at around 600^0C [7].

Reinforcing steel is much more sensitive to high temperatures than concrete. Both materials are incombustible but concrete has protective i.e. insulating role. Hot-rolled steels (reinforcing bars) retain much of their yield strength up to about 400^0C, but at temperatures >600ºC hot-rolled steel loses residual strength. Cold-drawn steels (prestressing strands) shows considerable loss of strength at 200-400ºC. Cold-worked steel loses residual strength at temperature >450ºC. Reducing the strength of reinforcement at high temperatures is usually the cause of the large permanent deflection of the structure.

When concrete is exposed to high temperature, as in the case of fire, the basic visible damages are thermal spalling and cracking, but other changes take place also, like a drop of strength and modulus of elasticity and change of colour. In most cases, a combination of these fire effects is registered.

Spalling is an umbrella term, covering different damage phenomena that may occur to a concrete structure during fire [4]. Spalling could be defined as violent or non-violent breaking off of layers or fragments of concrete from the surface of a structural element during or after it is exposed to high and rapidly rising temperatures as experienced in fires [13]. These phenomena are caused by different mechanisms [4]:

- Pore pressure rises due to evaporating water when the temperature rises;
- Compression of the heated surface due to a thermal gradient in the cross section;

- Internal cracking due to difference in thermal expansion between aggregate and cement paste;
- Strength loss due tu chemical transitions during heating.

There are several main theories explaining the spalling mechanisms [11, 12]:
- Thermal stress theory
- Pore pressure theory
- Combined pore pressure and thermal stress spalling.

During last few decades several specific theories were developed [11]:
- The fully saturated pore pressure theory
- The BLEVE theory (Boiling Liquid Expanding Vapour Explosion)
- The frictional forces from vapour flow theory

All of these theories are based on the phenomena of "the movement of heat and / or movement of moisture" that cause stresses. Unfortunately, mentioned theories have not been entirely confirmed by a number of experiments. The same conclusion can be derived for numerical modelling that attempt to explain and predict the occurrence of spalling.

Cracking of concrete exposed to fire occurs due to exceeding of concrete tensile strength. Cracks and fissures are caused by thermal expansion and dehydration of the concrete due to heating.

3. TYPES AND CLASSIFICATION OF DAMAGES

3.1. Types of damages

Term "**spalling**" encompasses large number of damage types. The first types of spalling were described in the beginning of the 20th century (explosive, surface, aggregate and corner spalling). Over the next decades two new types were added (sloughing off spalling and post cooling spalling) [4, 13]. They are:
- **Explosive spalling:** Violent breaking off of concrete fragments at high temperatures generally occurs in the first 30 minutes of a fire. Explosive spalling is usually caused by: insufficient release of high pore pressure, high thermal stresses and combination of both. This type of spalling is especially likely to occur on structural members heated from more than one side, such as columns and beams. When moisture clogs are advancing into the concrete from all heated sides, at some point in time the moisture clogs will meet in the centre of the cross-section, giving a sudden rise in pore pressure which may cause large parts of the cross-section to explode.
- **Surface spalling**: Violent separation of small or larger pieces of concrete from the cross section at high temperatures, during which energy is released in the form of popping off of the pieces and small slices with a certain speed. Usually occurs in the first 30 minutes of a fire.
- **Aggregate spalling**: Splitting of aggregates due to their decomposition or changes at high temperatures. Usually occurs in the first 30 minutes of a fire (Fig. 1).
- **Corner spalling**: Removal of concrete cover from corners at high temperature due to the temperature impact from two sides. This type of spalling is usually connected

with splitting cracks due to difference in thermal deformation between concrete and reinforcement and occurs in the first 90 minutes (Fig. 2).

- **Sloughing off spalling**: Sloughing off is the form of progressive gradual spalling, that is caused by strength loss due to internal cracking and chemical deterioration of the cement paste.This type of spalling is non-violent breaking off of concrete fragments after longer exposure to high temperatures, when concrete loses its strength (Fig. 3 and 4).
- **Post-cooling spalling:** Non-violent breaking off of concrete fragments during cooling from high temperature. This type of spalling was observed with concrete types containing calcareous aggregate. An explanation is the rehydration of CaO to $Ca(OH)_2$ after cooling, when moisture is again present on the concrete surface. The expansion due to rehydration causes severe internal cracking and thus complete strength loss of the concrete. Pieces of concrete keep falling down as long as there is water to rehydrate the CaO in the dehydrated zone (Fig. 5).

The term "**cracking**" covers the following types of damage:

- **Crazing**: Mesh like fissures and cracks on the surface of the concrete elements (Fig. 6), caused by additional shrinkage of hardened cement paste during drying due to high temperature.
- **Corner cracks along main reinforcement**: Cracking due to difference in thermal expansion/deformation between concrete and reinforcement bars. These cracks are usually located along the edge of columns and beams, especially in the direction of the main reinforcement. Also, they are associated with the separation and falling off of pieces of concrete (corner spalling) and with visible reinforcement bars (Fig. 2).
- **Inner delamination of concrete**: Is manifested as internal crack parallel to the fire-affected surface (Fig. 7). The main cause of this damage is high temperature gradient that induces high tensile stresses between the heated surface layer and colder inner zone of concrete. This phenomenon is typical for the columns. Since the internal cracks cannot register visually, their existence must be checked by extracting concrete cores.

Concrete surface cracking may provide pathways for direct and faster heating of the reinforcement bars and inner concrete, possibly bringing about more thermal stress and further cracking.

Loss of strength and ductility of reinforcement are usually consequences of high temperatures during fire. Visible characteristic fire damages of reinforcement are:

- Plastic deformations due to restrained elongation (Fig. 8).
- Breaking of bars (Fig. 8) due to loss of ductility of the steel or local reduction of bar cross section because of melting of steel.

Reinforced concrete elements during fire are subjected to additional stresses due to restrained deformations. In a case of slender beams and slabs buckling associated with deflection may occur. Under fire conditions, axially restrained beam/slab develops large deflections in post-buckling states [23].

Extent and type of described fire damages of RC structures depends on numerous parameters, among which the most important are: size and distribution of fire load, fire duration, fire maximum temperature, the shape and dimensions of structural elements, the existence and type of finishing layer - cover of the RC elements, the presence of defects and/or prior damage, construction details and the actual quality of concrete.

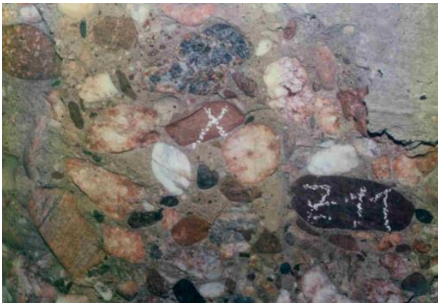


Figure 1. Aggregate spalling

Figure 2. Corner spalling and corner cracks along main reinforcement

Figure 3. Sloughing off spalling (beam)

Figure 4. Sloughing off spalling (slab)

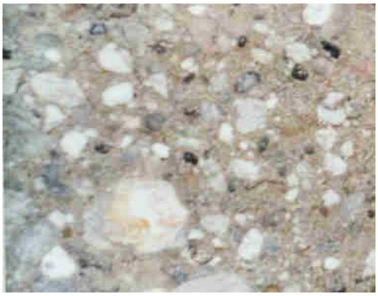


Figure 5. Post-cooling spalling

Figure 6. Crazing – mesh like cracks

Figure 7. Inner delamination of concrete in the column

Figure 8. Plastic deformations and breaking off of bars

3.2. Classification of damages

Among a numerous available classification of concrete fire damages authors of this paper chose the classification proposed by Ingham and Tarada [10] and modified it in relation to the degree of affected part of RC element cross section. Figure 9 illustrates parts of cross section of typical RC element that have to be considered during selection of appropriate repair method. Proposed classification is given in Table 1.

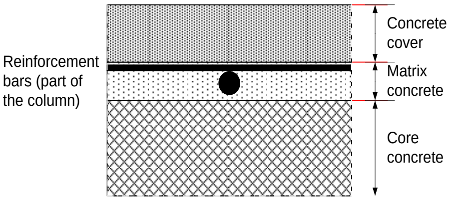
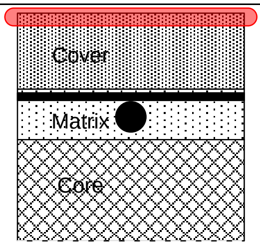
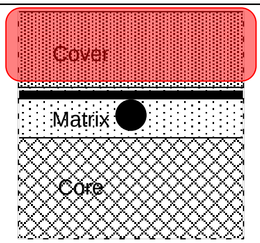
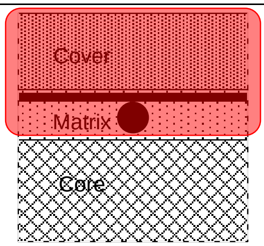
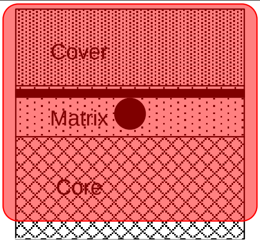


Figure 9. Characteristic parts of cross section of RC element

Table 1

Classification of fire damages with illustration of affected part of cross-section

Damage degree	Affected part of cross-section	Illustration	Features observed
1	Surface thin layer		Minor crazing – mesh like fissures with normal concrete colour Spalling is non-visible Rebars are non-visible
2	Concrete cover		Moderate crazing - mesh like cracks Surface spalling Aggregate spalling Change of concrete colour (pink or red) Rebars are non-visible or locally visible at places with insufficient cover (up to 25%)
3	Concrete matrix		Extensive crazing Corner spalling and cracks along rebars Sloughing off spalling Change of concrete colour (pink/red/whitish grey) Up to 50% of rebars are visible Loss of concrete strength Minor deflection of RC elements

4	Concrete core		Deep extensive spalling More than 50% of rebars are visible Change of concrete colour (whitish grey/buff) Possible melting of concrete (long-lasting fires) Inner delamination of concrete Impaired bond between concrete and rebars Increase of deflection of RC elements Reduction of rebars mechanical properties Possible buckling and breaking off of rebars

4. VULNERABILITY OF CONCRETE STRUCTURES EXPOSED TO FIRE

According to EN 1992-1-2 [5] fire resistance design of the construction requires three levels of analysis:

- Member analysis,
- Analysis of part of the structure, and
- Analysis of entire structure.

Besides heating rate, duration and maximum temperature of the fire, vulnerability of RC structures exposed to fire on member level depends on numerous additional factors, such as:

- Type of concrete (ordinary concrete, lightweight concrete, high strength concrete, fiber reinforced concrete, self-compacting concrete, etc.)
- Shape and dimensions of the structural members (simple/compact or complex cross sections)
- Construction and architectural details (concrete cover thickness, reinforcement arrangement, placement of installations)
- Presence of defects and previous damages and repairs (honeycombs, insufficient concrete cover, poor cold joints, cracks)
- Existence and nature of protective and decorative layers (combustible and incombustible layer materials).

4.1. Member analysis

4.1.1. Concrete type

Fire properties of *ordinary concrete (OC, NWC)*, as well as other types of concrete, mainly depends on aggregate type, as aggregates occupy 65-75% of the volume of concrete. In EN 1992-1-2 [5] aggregate is split in two groups, siliceous and calcareous aggregate. Calcareous aggregate has better fire properties than siliceous aggregate. However, some authors propose different siliceous aggregate classification. For example Khoury [14] divided siliceous aggregate in two groups, with better thermal stability (up to 600^0C) such as basalt, granite and gabbro and with lower thermal stability (below 350^0C) like flint and river aggregate. From the aspect of thermal stability, the least favourable aggregates are those obtained from rocks of metamorphic origin, primarily of quartzite rocks. Namely, quartzite rock contains a significant amount of mineral quartz, which is considered as the most critical mineral of solid rock at elevated temperatures [3]. Considering concrete behaviour at high temperature, besides thermal stability, a suitable aggregate would be one with a low thermal expansion, which improves thermal compatibility with the cement paste, rough angular surface, which improves the physical bond with the cement paste and the presence of reactive silica, which improves the chemical bond with the cement paste [14].

Lightweight aggregate concrete (LWAC, LWC). Concrete with artificial mineral lightweight aggregate, such as expanded clay, is the most frequently used type of LWAC for structural purpose. Structural lightweight concrete is advantageous in terms of reducing the dead load of the structures and the lateral earthquake loads. In addition to lower density, this type of concrete has a lower thermal conductivity and transmits heat at a slower rate than ordinary concrete. However, LWAC tends to have a reduced tensile strength compared to NWC, for the same compressive strength. The replacement of traditional aggregates by lightweight aggregate generally results in an increase in the occurrence of spalling for temperatures above 350 °C. The occurrence of spalling in LWAC is due to its lower tensile strength, higher moisture content and the development of higher thermal gradient during heat exposure. When spalling does not occur, the deterioration of LWAC due to elevated temperatures is similar to OC [25] or smaller, thanks to lightweight aggregate that has already been exposed to temperature >1000^{0}C during Pyroprocessing. In fact, the residual strength is generally higher in LWAC than in OC due to the higher thermal compatibility of the constituents of LWAC. Since, the difference between the coefficients of thermal expansion of the aggregate and the cement paste is higher in the OC than in LWAC, the OC is more prone to cracking. Also, LWC protects more efficiently the tension bars from the heat flow, with a remarkable increase of the bearing capacity at any fire duration [19].

High strength concrete (HSC). Thanks to technical and economic benefits HSC is increasingly becoming a key component in large-scale construction, from tall commercial and residential buildings to bridges, tunnels and offshore structures. The basic properties of HSC are high compressive strength and modulus of elasticity (stiffness), decreased permeability and abrasion resistance. Fire-exposed HPC has a different tendency and feature of spalling compared with OC. Due to the low permeability of HPC, which makes it more difficult to transport vapour and moisture, very high vapour-pressure may occur close to the surface [18]. This means that there is a greater risk that HPC spalls compared with OC. When the vapour zone moves to a certain distance from the hot surface, a maximum vapour pressure is created, and at greater distances the pressure decreases again. This critical distance is much less for HPC, about 5-10 mm than for OC, about 20-40 mm (Fig. 10). It has been observed from fire tests that spalling of HPC is characterized by a layer of about 5 mm of concrete falling off and after that a new vapour front buildup, which can create a new spalling of 5 mm, and in the end the total spalling can reach considerable depths [18]. The Compressive strength decrease of HSC begins at distinctly lower temperatures than that of OC. For example, at 150^{0}C compressive strength of HSC decreased to 70% of its room temperature strength, while reduction of compressive strength in OC is negligible. In HPC cement matrix and coarse aggregate are loaded with same stress level, because they have similar values of modulus of elasticity and compressive strength. At elevated temperatures cement matrix begins to weak before coarse aggregate and it causes redistribution of internal stresses and the stress is concentrated on the coarse aggregate alone which lead to significant reduction of the compressive strength of HPC. On the other hand, weakening of the cement matrix due to heating causes only slight stress redistribution and consequently only a little bit reduction of compressive strength of OC [8]. HSC exhibits brittle properties below 600^{0}C, and ductility above 600^{0}C.

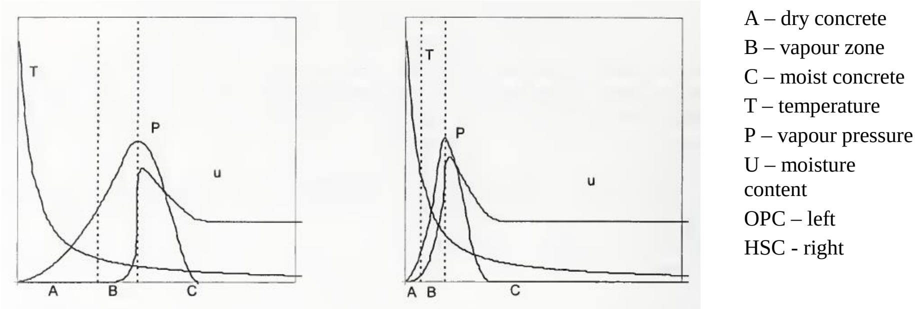


Figure 10. Illustration of water clog position and internal pore pressure within heated OC and HCC

Fiber reinforced concrete (FRC). Fiber reinforced concrete (FRC) is concrete containing fibrous material which increases its structural integrity. It contains short discrete fibers that are uniformly distributed and randomly oriented. Fibers include steel fibers, glass fibers, synthetic fibers and natural fibers. Steel, glass and other mineral fibers are incombustible, while organic synthetic and natural fibers are combustible. Among the mentioned fibers steel and polypropylene fibers are commonly used in concrete. So far a number of tests have been done to investigate the explosive spalling of concrete with PP fibers or steel fibers or steel/PP hybrid fibers. In FPC PP fibers are deemed to be effective in mitigating spalling risk under fire due to micro-channels generated by melting of PP fibers (Fig. 11) [26]. PP fibres melt at approximately 170^0C, and leave a network of open pores, which make steam evacuation easier, thus contributing to the reduction of internal pore pressure [24]. Eurocode had prescibred that 3kg/m^3 PP fibers can help avoid spalling. The diameter, length and amount of PP fibers have significant effect on reduction of spalling of concrete in fire, as well as on concrete compressive residual strength (Fig. 12) [27] For SFRC different conclusions were made about the effectiveness in mitigating explosive spalling under fire.

Figure 11. A view of OC (left - explosive spalling) and PPFRC (right) after exposing to elevated temperatures

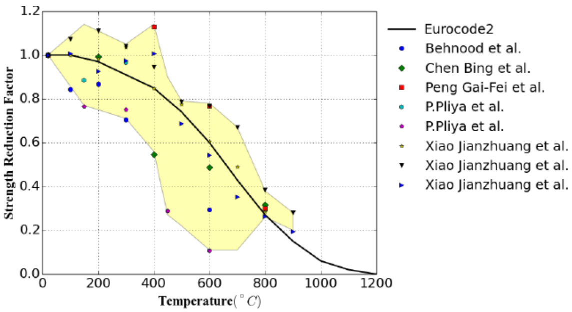


Figure 12. Compressive strength reduction against temperature for PP-FRC with (a) calcareous aggregates

4.1.2.Construction and architectural details

The arrangement of reinforcement in the structural members also has an impact on the degree of fire damage. Rebars with larger diameter and inadequate layout of reinforcement contribute to the intensification of damages caused by fire, especially in members with smaller dimensions or with insufficient cover (Fig. 14). Characteristic damages are falling off of concrete cover and plastic deformations of bared reinforcement.
Placement of electrical and similar installations within cross-section of RC elements frequently leads to significant local damage of concrete and reinforcement. The installations in buildings are usually placed in plastic pipes which melt and burnt during fire and realise additional heat and causes local damage of concrete core following with plastic deformation of rebars. Fig. 15 illustrates local severe damages of beam and slab, respectively, due to afterburning of installations in plastic pipes.

Inadequate sealing of holes for penetration of installations enables uncontrolled and fast fire spreading. Fig. 16 illustrates fast spreading of fire in both vertical directions caused by unsealed holes for installations in floor slabs and curtain façade wall without horizontal barriers.

4.1.3.Presence of defects and previous damages and repairs

Defects (concrete honeycombing, segregation zones, improperly executed cold joints, insufficient concrete cover, uneven edges, etc.) and damages (cracks) that existed before fire significantly increase the damage rate of RC members exposed to fire. In such zones, fire damages are spreading faster and deeper into the concrete mass and usually occupy concrete matrix and even concrete core. Characteristic damages are falling off of thick pieces of concrete, impaired bond between the reinforcement and concrete, plastic deformation of the rebars, as well as local reduction of concrete mechanical properties. Influence of improper executed cold joint in the pan joist on the degree of fire damages is shown in Fig. 17.

4.1.4.Existence and nature of protective and decorative layers

Inorganic mineral coatings of concrete surfaces (mortar, plasterboard, ceramic and stone tiles) play a very important role in protecting RC members during the fire. The advantages of these materials in fire are two-fold. They are incombustible and also good insulating materials possessing a low thermal conductivity. However, inorganic mineral materials are not refractory materials they will gain serious damages and even could be destroyed during the fire. Although these coatings are relatively thin (2-5 cm thick) they will prevent RC members from rapid heating and appearance of serious damages and increase their fire resistance. On the other side, organic coatings (wood, plastics, textiles, etc.) are combustible materials and contribute to local development of high temperature on the surface of RC members which leads to intensification of their fire damages. Influence of coating type on the fire damage degree of concrete columns is shown in Fig. 18.

Figure 13. Typical damages of pan joist floor system due to fire

Figure 14. Characteristic damages of improper reinforced rib due to fire

Figure 15. Local deep overheating of concrete and deformed rebar due to wirings within cross-section (a – beam, b – slab)

Figure 16. Spreading of fire in both vertical directions [9]

Figure 17. Influence of defects on fire damages degree in pan joist floor system

Figure 18. Influence of type of decorative layer type on the fire damage degree of concrete columns

5. METHODS FOR CONCRETE REMOVAL

Before beginning the repair and strengthening of the structure it is necessary to remove all additional loads and to support the structure. Besides preserving the stability of the structure during repair works, these activities are important in cases of structural repair where the new concrete is expected to carry its share of the load in the repaired elements. In the scope of repair very important role play proper selection of a method for concrete removal. Since there are a number of methods for concrete removal which differ in possibilities and limitations of application, it is not easy to select appropriate method. Depending on the damaged part of cross-section, authors of this paper propose following methods for concrete removal (Fig. 19).

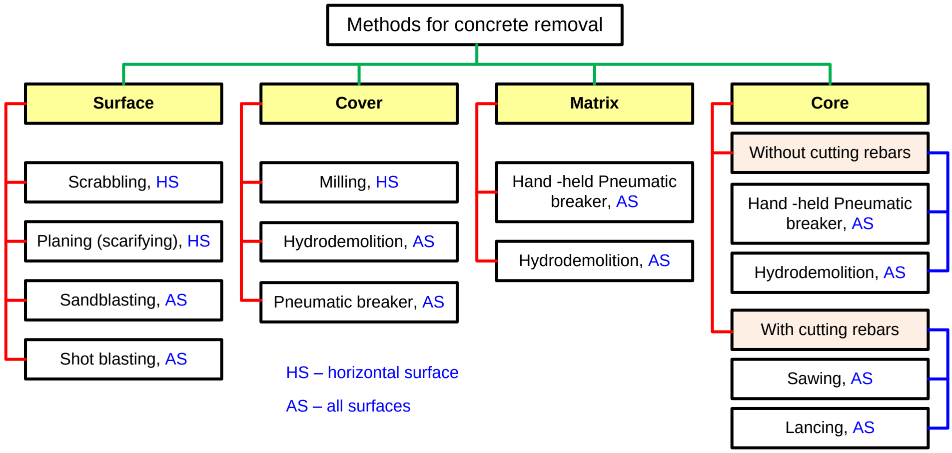


Figure 19. Suggested methods for concrete removal

6. SELECTION OF REPAIR METHOD AND MATERIAL

Based on the recommendations for repair of fire damaged RC structures in analysed literature [6], [21], [22] and on authors professional experience [15], [16], [17] decision about general repair strategy (structural or non-structural repair) mainly depends on affected part of the cross-section and state of the reinforcement. Non-structural repair is proper choice if rebars are not or locally visible. In all other cases structural repair is required, when: reinforcement is visible, bond is destroyed, rebars have plastic deformations, structural elements have excessive deflections etc. Structural repair is also mandatory in situation when all pointed out features are not accented but inner delamination of concrete exists. In some cases main reasons for structural repair is doubt regarding remaining structural capacity and intention to provide additional structural safety during future exploitation. For easier decision about type of repair method, Table 2 could be useful. An example of structural repair solution for damaged RC beam is shown on Fig. 20.

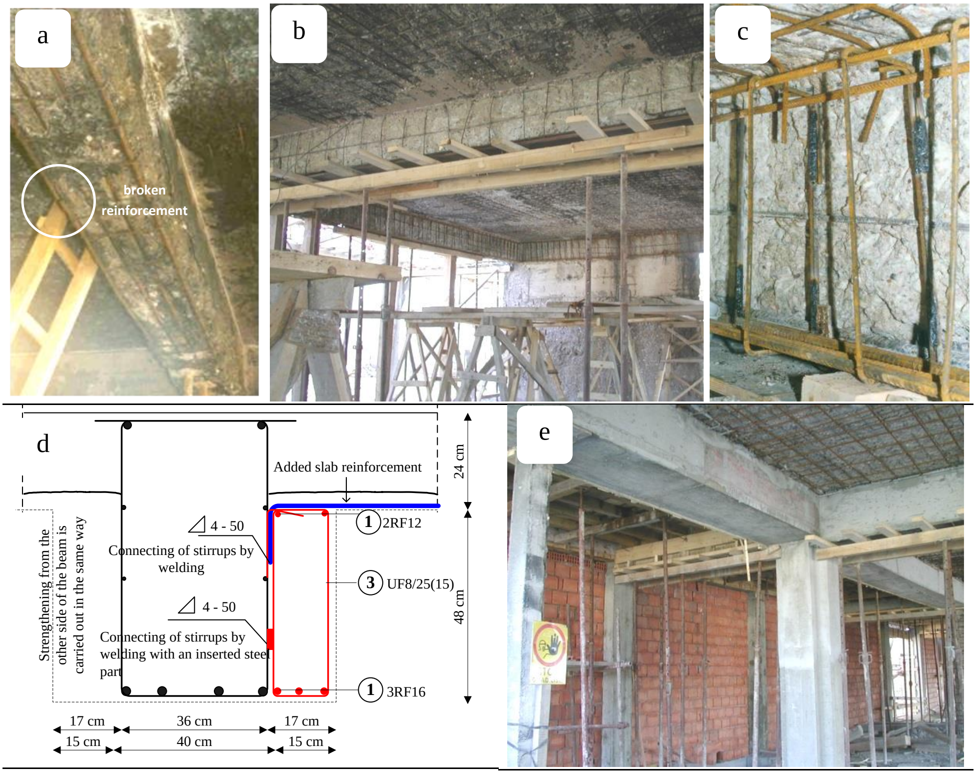


Figure 20: Strengthening process:
a) View of RC beam damaged in fire, b) Supporting of the beam and removal of damaged concrete, c) Instalment of new reinforcement, d) Detail of enlargement of existing cross section and arrangement of reinforcement, e) Vie of the beam after strengthening

Table 2
Suggested repair methods and materials

Damage degree	Affected part of cross-section	General repair method	Short description
1	Surface thin layer	Minor surface repair	Non-structural repair mortar (by hand)
2	Concrete cover	New concrete cover with/without light mesh	Structural mortar (applied by hand or spraying) Sprayed concrete with mesh
3	Concrete matrix	Structural repair and/or minor strengthening	Reinstatement of concrete cross-section with or without partial replacement of damaged rebars (flowable or sprayed concrete with mesh) Enlargement of cross-section and addition of new rebars (flowable or sprayed concrete)
4	Concrete core	Major strengthening or RC element replacement	Enlargement of cross-section and addition of new rebars (flowable or sprayed concrete) New RC element

7. CONCLUSIONS

The authors of this paper, through brief theoretical consideration of damage mechanisms of concrete and steel, classification of fire damages of RC structures and possible repair methods with respect to affected part of cross-section, tried to assist students to understand complex behaviour of reinforced concrete at elevated temperatures and to make decision about possible repair solution.

Through many years of experience in the assessment and repair of the structures due to fire, as well as on the basis of the analysis of the vulnerability of structural elements at the material level, at the level of member and through the analysis of the entire loadbearing structure, the authors of this paper concluded that RC structures in general have satisfactory fire resistance, but analysed influence factors, such as type of concrete, shape and dimensions of members, defects etc., could improve or jeopardise vulnerability of whole structure. On the other hand, composite or prestressed structures are more sensitive when to elevated temperatures compared to RC structures. When composite structures are designed or structural elements of different materials are combined, the vulnerability of the entire primary loadbearing structure depends on the vulnerability of the most sensitive structural member. Therefore all elements of the primary structure must have the same degree of vulnerability, which is achieved by the adequate choice of the structural system, the material for the structural members and the active fire protection measures

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ASSESSMENT AND REPAIR OF RC STRUCTURE DAMAGED IN A FIRE - CASE STUDY

Vlastimir Radonjanin, Mirjana Malešev

1. INTRODUCTION

In April 2000, a fire spread in the building of the Novi Sad Open University (Fig. 1 and 2). It broke out on the 12th floor and rapidly spread over the last six floors of the building. Due to the fire, which lasted for approximately six hours and was mainly extinguished with water, the load-bearing structure of the building was damaged and the facade, the interior and the installations were completely destroyed on floors caught by fire.

Figure 1. The 9th floor in fire

Figure 2. The view of the building after fire

The unfavourable structure of the building (non-existence of vertical fire resistant separating walls, non-insulated installation openings and staircases, etc.), a large amount of flammable materials in the building and the remarkably strong wind (13 – 15 m/s) enabled the quick spreading and instant development of the fire along both horizontal and vertical lines.

The fire destroyed six floors (8th to 13th), the total area of which was approximately 2.400 m^2.

2. BASIC DATA ON THE BUILDING

The building of the Novi Sad Open University was constructed in 1966. The building consists of the basement, ground floor, mezzanine and 13 floors. The dimensions of the basis (Fig. 3) of the building are approx. 30x15 m, and the height is 54 m. The height of the floors is as follows: basement – 2.6m, ground floor – 6.0 m, first floor – 6.0 m, floors from 2 to 13 – 3.5 m (cross-section on Fig. 4).

The structure of the building consists of a RC skeleton with an arrangement of columns at the distance of 6 m in both the longitudinal and transversal direction and RC walls in the staircase area.

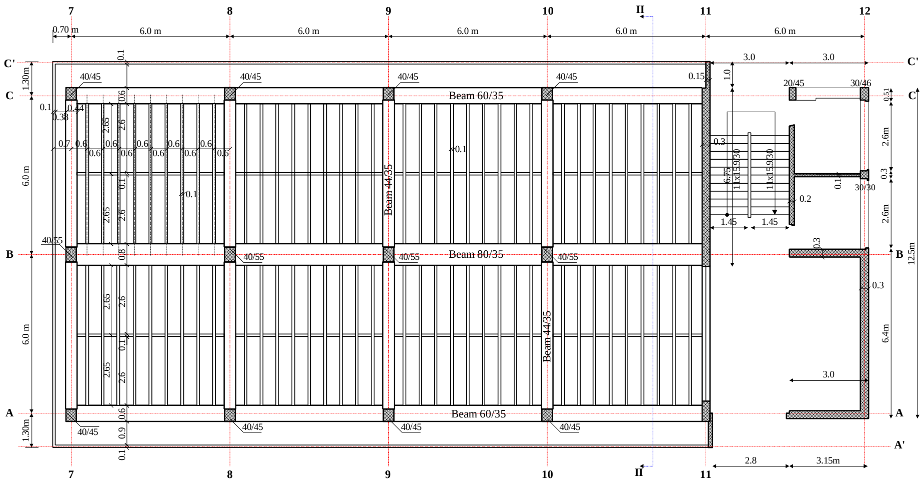


Figure 3. The plan of a standard floor

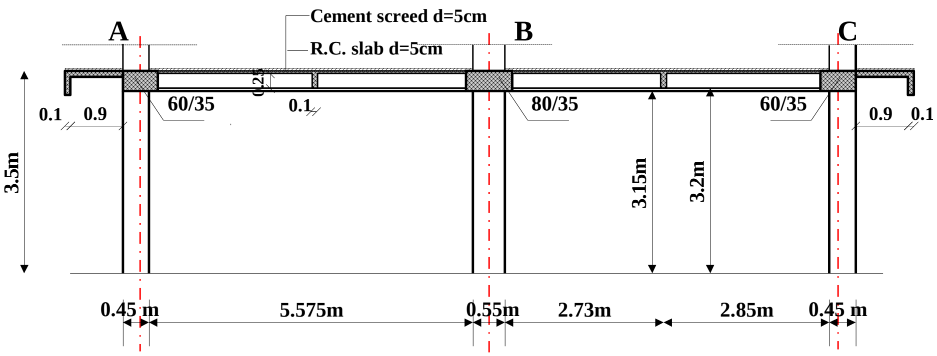


Figure 4. Cross-section of a standard floor

The basic elements of the load-bearing structure are:

- RC columns rectangular in cross-section, whose largest dimensions are in the basement, i.e. 50x90cm and 50x76cm; the smallest are the dimensions on the 13th floor, i.e. 35x40cm and 35x35cm.

- RC longitudinal beams, 6m in span, with a rectangular cross-section of variable dimensions, from the floors 8 to 11 - 80x35cm and 60x35cm, and on the 12th floor - 35x45cm.
- RC transversal beams, 6m in span, with a rectangular cross-section, 35cm high and of variable width, 44cm – 38cm from the 8th to the 12th floor.
- RC walls constructed along the whole height of the building in the staircase and elevator area, with the constant thickness of 30cm.
- RC walls of the basement and ground floor.
- RC pan-joist structure, a 5cm thick slab; the dimensions o the ribs are 10x25cm with a 60cm axial distance between the ribs.
- Solid RC slab over the basement and the mezzanine.
- RC facade beams, with dimensions of 10x32cm.
- RC two-flight stairs, built along the whole height of the building, with a 10cm thick slab.
- Piles and connective foundation beams.

The bearing steel structure of the facade of the building is connected to the RC structure. It is built on three sides of the building, from the third floor up to the top.

The partition-walls from the 3rd to the 12th floor are light partitions, consisting of a wooden frame wainscoted with chipboards. The columns are wainscoted with the same material. The lowered floor system consists of perforated boards on a wooden structure. The interior of the facade parapet is wainscoted with pressed chipboard. On the highest, 13th floor, the suspended ceiling is made of mortar on a wire lath. On this floor the columns are covered with marble plates set in mortar.

3. THE ASSESSMENT OF THE STRUCTURE AFTER THE FIRE

In order to assess the degree of damage to the bearing structure and facade of the building, as well as the kind and scope of repair work, the following records were made and tests carried out:

- Subsequent testing of the quality of built-in materials (concrete and reinforcement).
- Detailed survey of the RC structure and steel facade structure.
- Control calculation of the structure.

The subsequent testing of the concrete compressive strength was carried out using destructive and non-destructive methods. The number and the arrangement of the measuring spots were chosen with the aim of assessing the possible changes in the mechanical characteristics of the concrete caused by the fire. For that purpose, tests were carried out on the RC elements directly exposed to fire, as well as on the structural elements that were not exposed to fire or were protected by marble plates and/or mortar.

On each floor caught by fire, all the elements of the RC structure were surveyed in detail, with the focus on the following:

- Defects resulting from flaws in the construction phase;
- Damages caused by the fire.

3.1. The condition of the structural elements from the 8th to the12th floor

This section of the paper presents only the conclusions of the condition of the structure after the fire and the photographs of the typical damages to the individual structural elements. A detailed description of the conditions of the structure after the fire and all the results of the testing are presented in a previous paper [3], [5].

The general view of the characteristic floor after the fire and cleaning of residues of interior is presented in Fig. 5.

Figure 5. The view of the damaged structure on the 8th floor after the fire and clearing

The conclusions about the condition of the RC structure after fire were:

- Concrete compressive strength for columns, beams and curtain walls was ca. 30 MPa.
- Concrete compressive strength for the ribbed ceiling was ca. 20 MPa.
- The reinforcing steel had the mechanical characteristics corresponding to those of quality of mild reinforcement GA 240/360.
- The decrease in concrete strength (31-34%) and the preservation of the mechanical characteristics of the reinforcing steel confirm the theoretical insights on the change of the properties of these materials when exposed to fire.
- The reinforced concrete structure has been built with numerous faults (concrete honeycombing, uneven, frequently insufficient protective concrete layers, imperfections, irregularly breaks and continuation in concreting).
- All reinforced concrete columns were damaged in the fire (characteristic view in Fig. 6). The concrete cover, approx. 4 cm thick, is crumbled, dilapidated and cracked. In the interior of concrete, at the depth up to 12 cm, there are cracks which point to the separation of the outer layer from the sound core of concrete. The edges of columns are separated or fallen off up to longitudinal reinforcement. Bond between concrete and reinforcement has been impaired.
- All longitudinal and transversal reinforced concrete beams were damaged due to the fire (characteristic view in Fig. 7). The surface concrete layer from the lower side was dilapidated and broken off (thickness of this layer was approx. 5cm). The edges were cracked or broken off along the whole height, and longitudinal reinforcement was bared. There are vertical cracks on the sides of approx. 0.5 mm of width, and horizontal cracks at the joints of transversal beams and slab. Adhesion between concrete and rebars in the lower zone has been impaired.

- The pan-joist structure was the most severely damaged element of the load-bearing structure due to its small dimensions and unfavourable position in relation to the fire (characteristic view in Fig. 8 and 9). The lower parts of ribs up to 15 cm in height were broken off, while the remaining parts were cracked and dilapidated. The reinforcement was bared along the whole length of the ribs, and adhesion between it and concrete was impaired. There were slanting or vertical cracks at the points of swaying of reinforcement bars from the lower into the upper zone and horizontal cracks at the joint with the plate. The slab was 5 cm thick and had net-like cracks on the lower side. The 3 cm thick cover was dilapidated, crumbled and fallen off, especially at places of uneven changes in dimensions and breaks of concreting, where the damage affects the slab in its whole thickness.
- The reinforced walls in the axis 11 are damaged from the side which was not mortared, and which was directly exposed to fire. The concrete cover, approx. 4 cm thick, was dilapidated, crumbled and falls off, especially at the places of concrete honeycombing. There were net-like and vertical cracks. Reinforcement was partially bared.

Figure 6. Typical view of the column after fire

Figure 7. The characteristic view of the longitudinal beam after fire

3.2. The condition of the structural elements of the 13th floor

- The concrete compressive strength in the columns was ca. 60 MPa.
- The concrete compressive strength in the curtain wall in axis 11 was ca. 30 MPa.
- The columns have not been damaged by the fire. Some of them suffered only isolated surface damage.
- The curtain wall in axis 11 has been damaged only on the side of axis 10 in the same way as the walls on the lower floors.
- The beams and the ribbed ceiling have not been damaged.

Figure 8 and 9. The characteristic view of the pan-joist structure after fire

3.3. The condition of the façade steel elements

- The characteristic damage of steel façade elements was buckling due to restrained elongation. It was concluded that all façade steel elements from 8th to the 12th floor had been destroyed in fire.

After the analysis of all gathered data, it was concluded that the registered damages of the load-bearing RC structure from the 8th to the 12th floor jeopardized the stability and the load-bearing capacity of this part of the building. However, it was concluded that damaged structure could be repaired.

4. REHABILITATION AND STRENGTHENING OF MAIN STRUCTURE ELEMENTS

This chapter presents the chosen repair measures, which were selected according to the degree and the type of the damage and the type of the element of the RC bearing structure [4], [7].

4.1. RC Columns

In the choice of the repair measures for the RC columns on the 8th to 13 the floor, the presence of cracks in the column interior and the condition of the exterior concrete coat have been taken into consideration. Also, the replacement of the damaged concrete cover and the strengthening of the columns by adding a new longitudinal and transversal reinforcement were foreseen.

The repair measures consist of:

- The removal of damaged, dilapidated and cracked parts of concrete up to the sound concrete and uncovering the main reinforcement (Fig. 10 and 11),
- The placing of additional vertical reinforcement bars ∅ 14mm and stirrups ∅8mm/10cm (Fig. 12 and 13),
- Connecting the new bars with existing reinforcement by direct welding and additional steel plates (Fig. 12),
- Coating of the surface of the existing concrete and reinforcement of columns with the material for improving the bond between old and new concrete,

- Applying a new concrete cover with polymer modified concrete of class C40/50 in two phases through holes in the formwork from the upper slab (Fig. 14, 15 and 16).

By this method of rehabilitation and strengthening of damaged columns, existing cross-section dimensions were increased by 10cm.

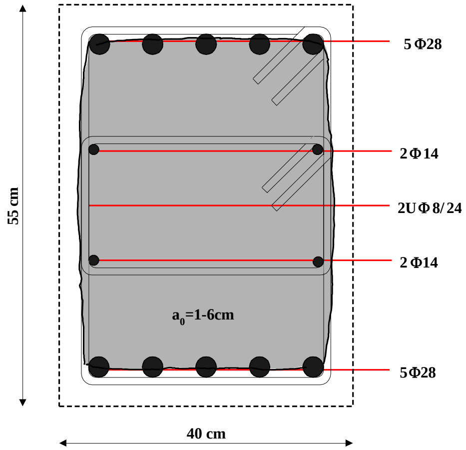
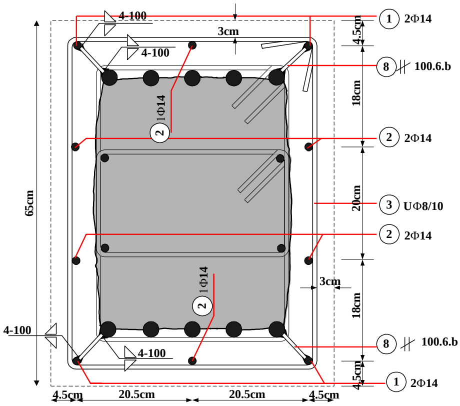


Figure 10 and 11. Removal of the damaged concrete layer from the column



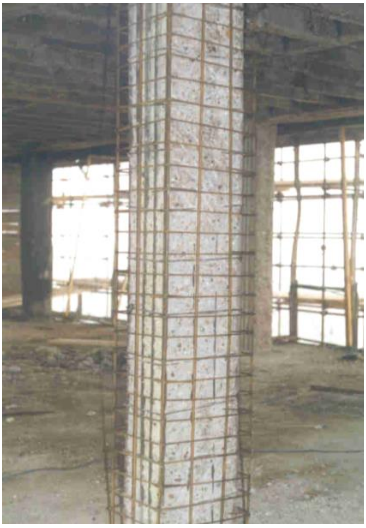
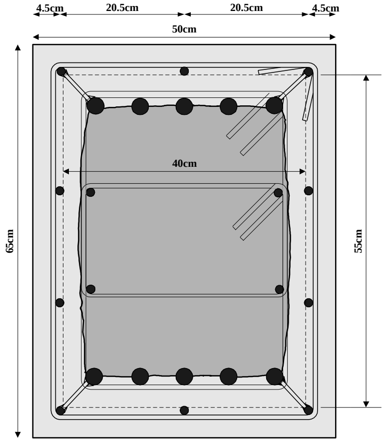


Figure 12 and 13. The placing of the new column reinforcement

Figure 14. Placing of the new layer on the column



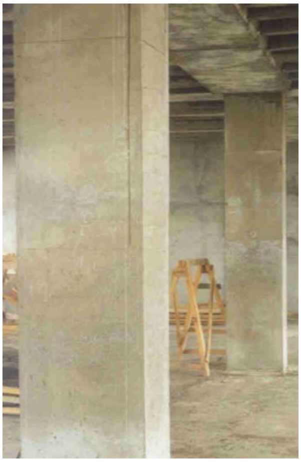


Figure 15 and 16. The repaired column

4.2. RC Beams

Since the damages of the RC beams due to the fire occurred only in the concrete cover, the reinforcement was not been damaged, these elements were repaired by replacing the damaged concrete cover.

The rehabilitation of RC beams consisted of:

- The removal of damaged, dilapidated and cracked concrete cover from bottom and side surfaces (up to the sound concrete and partial uncovering the main longitudinal reinforcement and stirrups (Fig. 17 and 18),
- Placing of the new cover by pouring self-levelling repair mortar through hollows in the slab (Fig. 19 and 20),
- The upper part on the side surfaces of these beams was repaired by a manual application of polymer modified cement mortar (Fig.21).

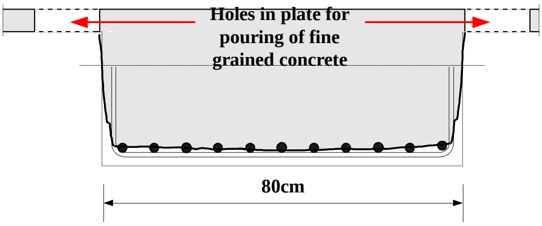


Figure 17 and 18. Removal of the damaged concrete cover and drilling a slab opening

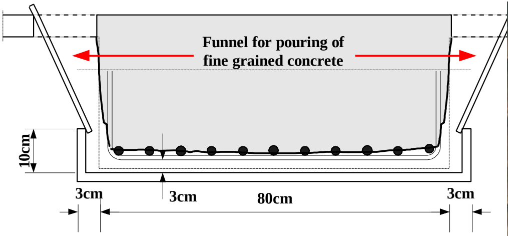


Figure 19. Placing of new cover on the down part of the beam

Figure 20. View of the new protective cover on the down part of the beam

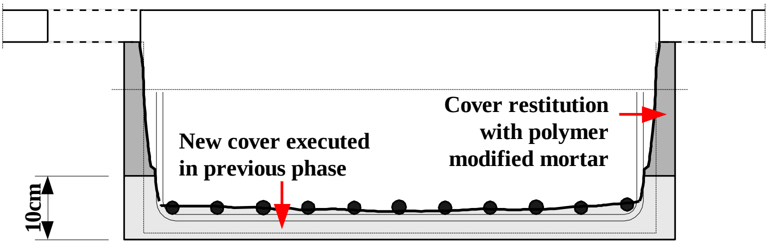


Figure 21. Applying the new protective cover on the side surfaces of the beam

4.3. Pan-joist Structure

RC pan-joist structure was the most damaged element of the structure. The required repair and reinforcement of the plate floor structure with the preservation of the same construction system would demand hiring a large number of construction workers who would have to repair manually almost all the ribs and most of the slab. Therefore, the repair measures where the existing floor structure is substituted without removing it, i.e., the building of new RC beams along the 1/3 of the span and a 6cm tick slab over the existing one (Fig. 22) was chosen as a solution. Figures 23 and 24 show the transversal and typical cross-sections of the new beams with the arrangement of the reinforcement.

The order of repair works is shown below:

- The removal of damaged, dilapidated and cracked parts of concrete from the ribs and the bottom surface of the slab and removal of screed;
- Supporting of the existing pan-joist structure ribs;
- Removal of parts of the slab for the execution new RC beams (Fig. 25 and 26);
- Installing of new beam reinforcement by anchoring in the existing longitudinal beams and placing the mesh reinforcement for the new slab (Fig. 27);
- Concreting of the new beams and the slab with concrete of class C30/37 (Fig 28.).

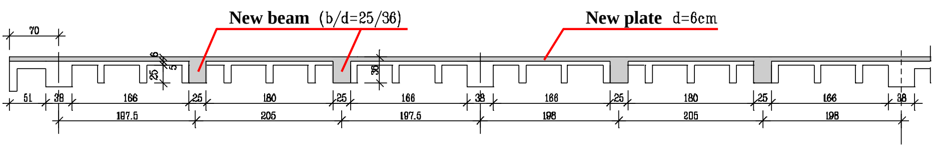


Figure 22. The arrangement of the new RC beams and the slab in the existing floor structure

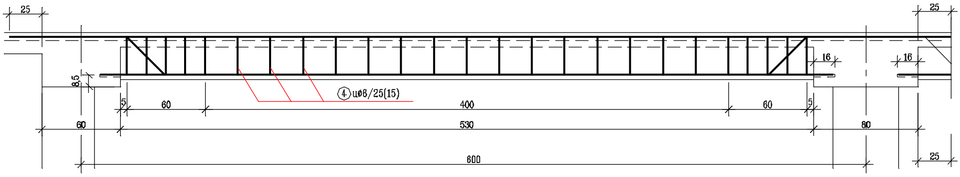


Figure 23. The new beam reinforcement plan: longitudinal cross-section

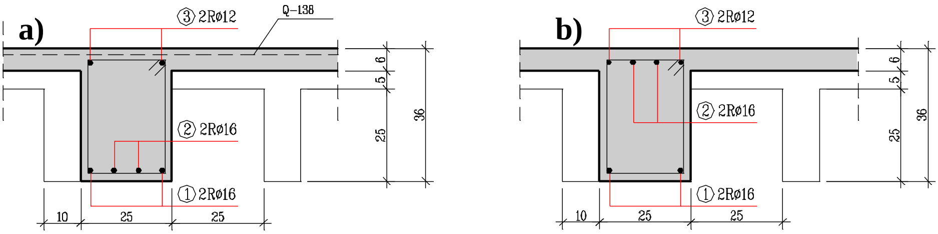


Figure 24. The new beam reinforcement plan: transversal cross-sections: a) in the span; b) in the support zone

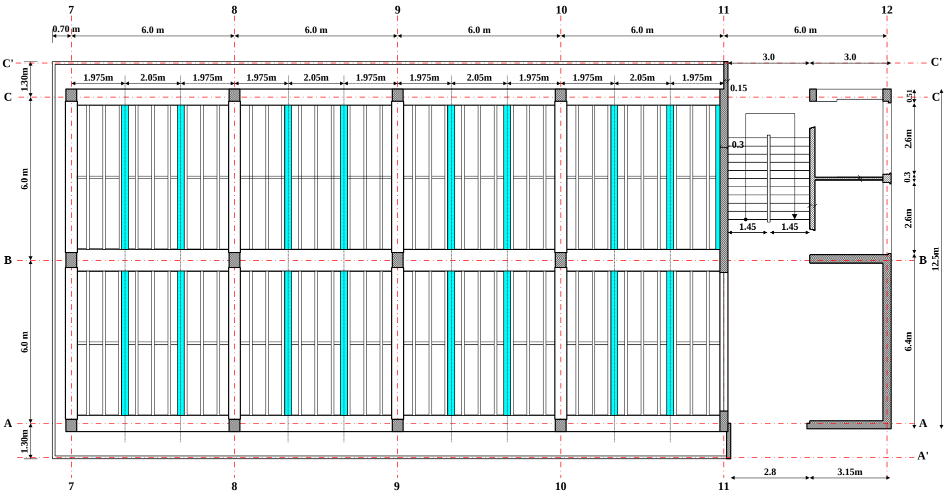


Figure 25. Disposition of the new beams in the existing slab

Figure 26. The reinforcement of the new beam

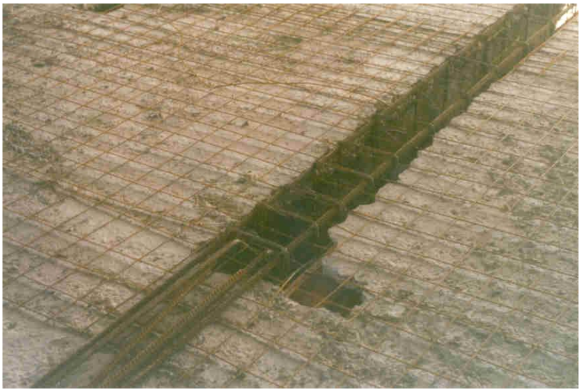


Figure 27. The wire-mesh reinforcement in the slab

Figure 28. Concreting of the new slab

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Section III

Western Balkan Legislation and Case Studies

PERSPECTIVE ON FIRE RISK MANAGEMENT IN THE BALKANS

Jelena Marković, Edisa Nukić

1. BOSNIA AND HERZEGOVINA, CASE STUDIES

Three characteristic case-studies of recent fires in Bosnia and Herzegovina, having different causes and sources of ignition, are described. Two fires, that occurred in shopping centres, were analysed. According to experts' reports, the first fire was caused by electrical wiring and failure of an appliance, and the second one was set on intentionally. A fire in a residential home was a result of methane explosion due to the sewage and septic tank improper installation.

1.1. Fire at the "Bingo" shopping centre, Lukavac Municipality

On 27th of July, 2014, at about 12:28, a fire started in the storage area in the "Bingo" shopping centre, Municipality of Lukavac. The storage area was completely destroyed by fire (Figure 1). The fire started in small tubular fan, located in the upper part of the wall, between storage and sanitary area (toilet).

Fire investigation determined that the cause of the fire was overload or indirect (incomplete) short-circuit on the Cu conductor of the connecting cable (P / L-2'0.75 mm²) for tubular fan located in the upper part of the wall between storage and sanitary area (toilet) [1].

There were no casualties in the fire, nor explosions, either in the vicinity or in the building affected by fire. The loss in property and the scope of destruction caused by this fire was very large.

Figure 1. Fire in the "Bingo" shopping centre in Lukavac

The details on the occurrence and development of the fire are shown in Figure 2, i.e. video footage taken by the surveillance camera located in the storage damaged in the fire. The video was obtained from the "Flek" security agency in charge. The place of fire's start and its development are easy to be observed in this video.

Figure 2. Details on start and development of fire

A large number of fires in residential, commercial and public facilities, both in the world and in Bosnia and Herzegovina, are caused by electrical installations failure. Since the number of facilities built in Bosnia and Herzegovina constantly increases, the total number of electrical consumers in them is increased as well. As a result there is an increase in number of locations where the electrical installation failures can cause fire. In addition, there are electrical products of low quality available on the market in Bosnia and Herzegovina, having characteristics that do not match with the one on products' declarations. Also, a large number of unauthorized individuals carry out repairs and adjustments in the electrical installations and appliances. Therefore the probability of fire due to electrical installations in B&H is higher than in countries with more strict legislation.

The above mentioned implies the need for more strict legal regulations in Bosnia and Herzegovina in regard to the following: the criteria for the periodical inspection of electrical installations, the license issued for offering electrical products on the market and the penalties imposed on individuals who perform unauthorized repairs and adjustments on the electrical installations and appliances.

In addition, it is necessary to organize programs aimed at education of the population also learning about possible consequences of non-compliance with the legislation in the related area. These should be joint efforts of research institutions in order to gain knowledge about the most often cause of fire resulting from electrical installation failures and on the basis of which the legislation improvement will be dealt with.

1.2. Fire in the "Škafa" shopping centre, Ilidža Municipality

On July 27, 2014 about 22:50 a very destructive fire started in the business area of the "ŠKAFA" shopping centre, located in the Municipality of Ilidža. Cause of fire: the fire belongs to the category of arson or intentionally set fire [2].

There were no casualties in fire, nor explosions in either vicinity or in the facility affected by fire. The "ŠKAFA" shopping centre was completely destroyed by fire.
Facility and its facade after fire are shown on Figures 3 to 6.

Figure 3. Entrance to Hall 1- from the street

Figure 4. Shopping centre's right facade in regard to main entrance

Figure 5. Shopping centre's left façade in regard to main entrance

Figure 6. Entrance to the Hall 2 basement floor (the opposite side of the main entrance)

Upon detailed observation of the shopping centre, particularly in the area of the fire site, it may be concluded that the complete workspace of the shopping centre, i.e. Hall 1, Hall 2 and their steel-structured connection area, formed one fire compartment.

The Fire Protection Study cites that fire protection door, with 60 minutes of fire resistance, is to be installed in the Hall 2, at the level of 4.70 m. Taking into consideration the fact that all indoor spaces of shopping centre make one large air space, this fire protection door has no significance and there is no proof that it has been installed even though there is an attestation of their fire resistance.

Fire protection door was not designed correctly by the Fire Protection Study. The door was not planned to be installed into a firewall whose dimensions should exceed the outside dimensions of the building. Also, the door was not completely separating the fire compartments in the shopping centre. Therefore, the installation of the fire protection door defined by the Study would be useless.

The connecting structure between Hall 1 and Hall 2, in which a large quantity of fabrics had been stored, was the centre of the fire, Figures 7, 8 and 9.

Figure 7. The zone of fire centre – observed from the Hall 2 basement

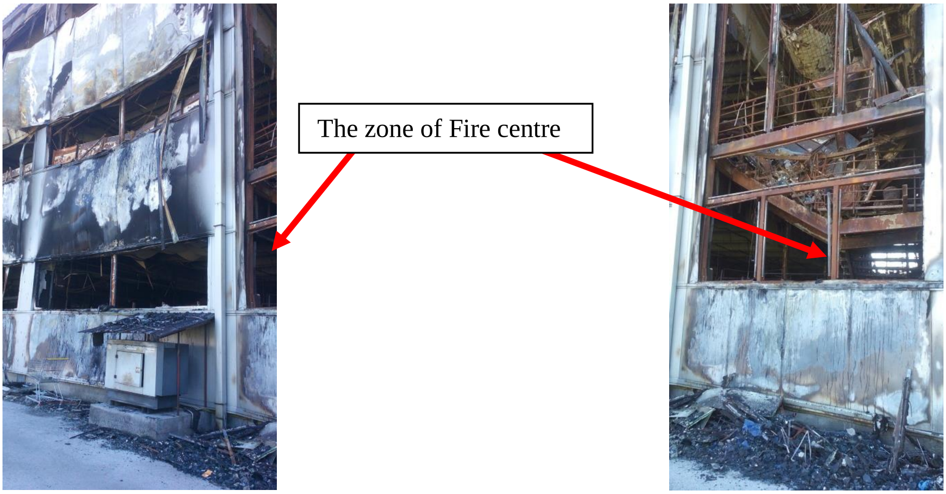

Figure 8. The zone of fire centre – observed from the outside, i.e. shopping mall facade

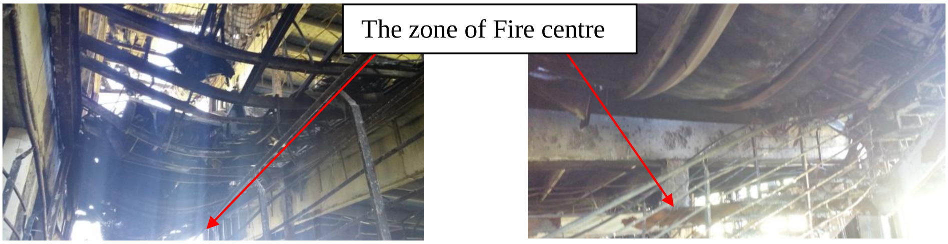

Figure 9. The zone of fire centre – observed from the Hall 2 basement

Considering that the centre of the fire outbreak was in the area of structure connecting two halls, should the wall with fire protection door was designed properly, as fire protection wall, the fire would not spread from the centre of an outbreak to Hall 2 for the 60 minutes of the fire protection door's resistance.

The shopping centre owner’s omissions that lead to destructive fire were as follows:

- The whole interior of the shopping centre was one fire compartment,
- There were no automatic fire extinguishing installations in the building,
- There were no fire-alarm installations in the building, that would inform fire department timely,
- The flammable material was stored in the building without the fire zone of separation for protection,
- The fire department was not informed on fire in time (a security guard’s cell phone was not operational),
- After receiving info on illegal entry in the building, the security personnel did not inspect whole interior of the shopping centre, but only the entrance door.
- The fire extinguishing was ineffective due to the lack of water needed for fire extinguishing, etc.

All the conditions were fulfilled for fire to develop to the extreme limits within the fire compartment, i.e. facility's external structure, and further.

1.3. The explosion of methane from a septic tank in the apartment on the 4th floor of the residential-business building in Bijeljina

In the sewage system, septic tanks particularly, the aerobic and anaerobic degradation processes of transported and deposited waste products occur. Aerobic degradation occurs in the presence of air and is characterized by the carbon dioxide release, while anaerobic degradation is caused by the activity of the microorganisms in the septic tank without the presence of oxygen. These processes are characterized by the dominant release of methane and carbon dioxide and a slightly smaller amount of ammonia. Due to the presence of these gases, when the sewage system is subjected to incorrect exploitation, there is a potential danger of the flammable and explosive gas mixtures occurrence.

On May 1, 2010, at about 19:20, a large explosion of methane occurred in a residential - business facility in Bijeljina, which caused significant damage in apartment No. 9. The accumulation of methane was the cause of the explosion in this apartment's bathroom, as a result of incorrect sewage and septic tank's installation. The investigation of septic tanks determined that the pipe immersed into the content of the septic tank enabled wastewater to flow into the tank due to the velocity of flow and gravity. It prevented the ventilation of this pipeline. As a result, biogas was accumulated in the sewage pipe filled with the content of septic tank.

A gas explosion in the bathroom was created right after the electric heater was switched on. The heater was placed on a washing machine behind the bathroom's entrance door. When the owner of the apartment entered the bathroom, a part of the biogas ran out, and that initiated the movement of the biogas layer in the bathroom and creating conditions for biogas to come into contact with an electric heater. The temperature of the heater in the appliance was significantly higher than the ignition temperature of the biogas with the dominant methane content, which was ignited and exploded having devastating effects of limited intensity.

There were no casualties in the explosion. However, the property endured substantial damage.

Figure 10. The apartment where the explosion occurred and the location of septic tank

The explosion had a substantial devastating force with high pressure at the front part of the impact airwave generated in the explosion process. The impact airwave broke the door of the bathroom, the apartment entrance door, as well as the room door. In addition, the explosion power caused displacement of the wall tiles in the bathroom, damage on the partition wall between the bathroom and bedroom, and deformation of door frames in the bathroom and bedroom. Damped impact airwave, which caused the partition wall between the bathroom and bedroom to crack, also caused the rotation of the closet leaned along the wall.

The pressure produced at the head of the air impact broke the bathroom and bedroom doors. The doors stopped in places where structural elements of the apartment and furniture prevented their further movement. The entrance door of the apartment landed at the top of the staircase leading to the upper floor of the building.
The explosion effects are shown in Figures 11 and 12.

Figure 11. Partition wall crack and bedroom door rotation caused by blast's air wave

Figure 12. Bathroom damaged door opening and bedroom door frame

According to explosion effects, it was determined that in the moment of contact of biogas with heater's hot wire, a "swell" of the biogas and air mixture occurred. The concentration of flammable gases in the bathroom was low, close to the lower explosion limit of 6%. Their combustion was short-lived, as much as the electrical heater was on. The short-term combustion occurred in the presence of excess oxygen, in comparison to the stoichiometric quantity required to produce carbon dioxide and water. Therefore the usual combustion effects, such as the formation of soot, were absent. The thermal effects of combustion due to high temperature were not noticed, which seems logical since the combustion was short-lived in a room with walls covered with smooth ceramic tiles. The heater stopped working due to a short circuit and the ignition source of the biogas and air mixture quickly disappeared.

A direction of the shock elastic wave implies that the explosion centre was in the apartment's bathroom, from where following the least resistance lines, it acted as destructive down the hall, and further through the bedroom, living room and entrance doors to all the sides.

The impact elastic wave's intensity dropped by doors' fracture of and an increase in the rooms' volume in which the wave had spread. Its intensity decreased to the limit at which the pressure at the front of the elastic impact wave reached the level of air pressure under normal conditions.

This phenomenon, in stronger or weaker intensity, with greater or lesser destructions, and even with more severe consequences, may occur at any time and any place where the sewerage system and septic tank are set up in the way it was performed in this residential-business building.

2. LEGISLATION IN BOSNIA AND HERZEGOVINA

European Union legislation EU 89/106/EWG of December 21, 1988, was to define the guidelines and application of EU Directives in the trade of construction products - CPD (Construction Products Directive) has been replaced by the European Parliament and EU Council Directive on March 9, 2011. For construction products - EU CPR 305/2011 (Regulation on Construction Products), that came into force on July 1, 2013 [4].

Federation of Bosnia and Herzegovina adopted the CPD requirements by promulgating the "Decree on buildings' technical properties concerning safety, application, and maintenance".

Upon promulgation of Regulation on Construction Products - EU CPR 305/2011, it is necessary to concur domestic legislation in that area:

- "Law on Construction Products", and
- "Decree on buildings' technical properties concerning safety, application, and maintenance",

with requirements and guidelines of the EU CPR 305/2011, to regulate the domestic market of construction products and works, and thus enable manufacturers from Bosnia and Herzegovina to export their products to the EU market and beyond.

The legislation of the Federation of Bosnia and Herzegovina related to facilities' fire protection, in addition to the Decree which accepts the European Directive on the technical properties which buildings need to satisfy in terms of safety or the ability to withstand foreseen capacities, while retaining all essential technical properties during the planned period of time, includes the fire protection.

Valid provisions of international and the accepted BAS standards are incorporated in four Regulations and one Decree. The Methodology regulates the content and method of fire vulnerability's assessment for cantons and municipalities, i.e. cities and legal entities.

Legislation regulating the area of fire protection is issued at the level of the state of Bosnia and Herzegovina, two entities, the Brčko District of Bosnia and Herzegovina and 10 cantons.

At the level of the state of Bosnia and Herzegovina, this area is regulated by "Framework Law on the Protection and Rescue of People and Property in the Event of Natural or Other Disasters in Bosnia and Herzegovina" (Official Gazette of Bosnia and Herzegovina 50/2008). At the level of the Federation of Bosnia and Herzegovina it is regulated by the following:

- Law on Fire Protection and Firefighting (Official Gazette of Federation of Bosnia and Herzegovina, 64/09) (in addition to the Law at the Federation level, the Cantons issue their laws on fire protection and firefighting for their area),
- Rulebook on the protection of high buildings from fire (Official Gazette of Federation of Bosnia and Herzegovina, 81/11),

- Rulebook on conditions, fundamentals and criteria for the classification of buildings to fire vulnerability categories (Official Gazette of Federation of Bosnia and Herzegovina, 79/11),
- Rulebook on technical standards for indoor and outdoor hydrant network for fire extinguishing (Official Gazette of Federation of Bosnia and Herzegovina, 87/11),
- Rulebook for fire protection of public buildings (Official Gazette of Federation of Bosnia and Herzegovina", 86/11),
- Rulebook on selecting and maintenance of fire extinguishers intended for the initial fire, that may be provided for a market with guarantee and service deadline included(„Official Gazette of Federation of Bosnia and Herzegovina", 46/11),
- Rulebook of conditions related to fire routes for access and passing for the residential and other buildings, structures and areas considered as constructions („Official Gazette of Federation of Bosnia and Herzegovina", 70/12),
- Rulebook on the scope and procedure in testing accuracy and functionality of the fire protection system installed, conditions to be met by legal persons conducting the test of accuracy and functionality, as well as the program and method of conducting the professional exam for these tasks („Official Gazette of Federation of Bosnia and Herzegovina", 69/13 and 2/18),
- Rulebook on the content and method in performing inspection supervision in the field of fire protection and firefighting under the competence of the Federal Civil Protection Administration („Official Gazette of Federation of Bosnia and Herzegovina", 22/16),
- Decree on the content and method for creating protection and rescue plans from natural and other disasters and fire protection plans („Official Gazette of Federation of Bosnia and Herzegovina", 87/11),
- Methodology for fire vulnerability assessment („Official Gazette of Federation of Bosnia and Herzegovina", 8/2011).

Rulebook on the protection of high buildings from fire regulates the appropriate planning, organizational, architectural, construction and technical-technological measures of fire protection for high buildings. These measures reduce the possibility of fires and, at the fire beginning, enable the safe evacuation of people and property and prevent fire spread. The Rulebook regulates measures for the development of project-planning documentation in accordance with the legal regulations on fire protection (development of fire protection projects, development of fire protection study and expert assessments on fire protection measures application to project documentation, creation of drafts for fire protection regulations, producing estimates for fire protection's vulnerability, development of fire protection plan).Fire and access routes, fire sectors, fire resistance of construction products, safety stairs, evacuation routes and fire-alert signalization are defined as a part of architectural-construction measures.

Technical-technological measures include at least all regulated measures from the point of view of mechanical and electrical installations and equipment installed in the building and are applied for preventive protection against fire and explosion. Such are: fire doors, blinds, shutters, ducts, smoke-release installation, safety elevators, boiler room, regular and spare electrical power supply, electrical installations and systems for early prevention and alert against fire and gas leakage, active and passive fire extinguishing systems using water, foam, aerosols, etc.

Rulebook on conditions, fundamentals and criteria for the classification of buildings to fire vulnerability categories regulates the categories of buildings, conditions, fundamentals and criteria for the classification of buildings and building sections into categories of fire vulnerability and the minimum statutory preventive measures for protection against fire and explosion are defined.

Rulebook for fire protection of public buildings regulates adequate planning-organizational, architectural- constructing and technical-technological measures for fire protection of public buildings that reduce the possibility of fires and, at the time of its creation, enable the safe evacuation of people and property and prevent fire spread.

Rulebook of conditions related to fire routes for access and passing for the residential and other buildings, structures and areas considered as constructions regulates the conditions that need to be fulfilled by firefighting access routes to the building or construction in order to enable firefighting technique to reach an opening in the external wall for people rescue and fire extinguishing.

Rulebook on technical standards for the outer and inner hydrant fire extinguishing network regulates technical standards when installing the hydrant fire extinguishing system in an aim to protect buildings and/or areas, types of fire extinguishing, networks' maintenance and calculation of the required amount of water.

Decree on the content and method for creating protection and rescue plans from natural and other disasters and fire protection plans defines the content, method of creating, a procedure for coordinating, accepting, updating and preservation of the plans for protection and rescue of people and property from natural and other disasters, and fire protection plans in the Federation of Bosnia and Herzegovina. The plan of protection and rescue defines the organization and method of implementation of protection measures. In particular, those measures are as follows: the application of building regulations (urban-spatial and technical documentation, etc.); technical measures for the construction of buildings in complex areas; construction of protection structures against landslides, floods (embankments, reservoirs, retentions and similar flood protection facilities), and execution of works on the streams and river regulation, spate regulation, support walls construction.

Methodology for fire vulnerability assessment regulates the content and the method in producing the assessment of vulnerability to fire for cantons and municipalities, i.e. town and legal entity, and the process of coordination, updating and preservation of the fire vulnerability assessment on all the levels during its creation.

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FAÇADE FIRE SAFETY – THE LEGAL FRAMEWORK IN WB AND EU COUNTRIES

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1. THE IMPORTANCE OF FAÇADE FIRE SAFETY AND HARMONIZED FIRE SAFETY REGULATIONS

Fire safety is one of the basic requirements buildings should meet during exploitation period. According to World Fire Statistics, in the 2017, the highest percentage of total fires were building fires (37.3%) [1]. The data on residential multi-storey building fires was collected in Finland and Sweden, for the 2004-2012 and 2004-2011 period, respectively. The results showed that out of average 508 fires per year in Finland, 10% percent of them started as external ignitions. In Sweden, the average number of fires per year was 2739, with 9% of external ignition fires. Furthermore, the statistical analysis of building fires reported in United States showed that out of 177,833 fires, 68% were residential fires. Although exterior wall fires (i.e. fires where external walls are the first ignited construction element) cause of 1.3% - 3% of total building fires and 2% of residential fires, due to the great extent of fire spread and injuries caused by combustible façade systems, in recent years, façade fires took dozens of human lives and caused great material damage [2]. The unfortunate incidents around the world confirm above stated - Tamweel Tower in Dubai, United Arab Emirates (2012), Mermoz Tower, in Roubaix, France (2010), residential building in Dijon, France (2010), residential building in Shanghai, China (2010), etc. [2]. The most recent event with disaster consequences was the Grenfell Tower fire in London, UK (2017), causing 72 fatalities [3].

The performance of building materials, especially the reaction to fire, which are part of the exterior walls of buildings, may significantly affect the possibility of the fire spread on the façade of the building, as well as the transfer of fire to adjacent rooms in the building affected by the fire. The minimum requirements regarding the design of façades are defined differently in various countries. While the general goal is the same - to reach the required fire safety level, the structure and practical application of the legal framework, as well as level of requirements development are different from one region to another, or even from country to country.

In order to define the minimum fire safety requirements that must be implemented into national legislation of EU member states, European Union (EU) sets the Regulation on Construction Products (CPR) [4]. Still, previous research studies on EU countries regulations revealed a significant differences in functional and performance requirements in European countries as well [5], [6], even after the implementation of CPR and development of Euro Codes. Many of those regulations are not completely the same, due to the fact that they had to be adjusted to the national specificities. The Western Balkan countries are mostly in the process of joining the EU and therefore are in transition period between the national and EU legislation. The countries that are in EU are implementing the EU regulations, although not to the same extent. This book chapter aims at analysing the regulatory systems in Western Balkan countries through the comparative analysis of fire safety of façades, as an important factor in building fire safety. The differences should point out the "weak spots" of implementing the regulations in practice, while the similarities should mark the potential of future collaboration and development of fire safety regulations in the same manner for whole region. This is of great importance, having in mind that certain design and construction companies and construction products are present in the markets of all Western Balkan countries. The comparative analysis was done on legal requirements in Serbia and EU Ex-YU countries Croatia and Slovenia.

2. THE CASE STUDY: COMPARATIVE ANALYSIS OF FAÇADE FIRE SAFETY LEGAL FRAMEWORK IN SERBIA, CROATIA AND SLOVENIA

Serbia, Croatia and Slovenia have implemented the CPR in national legislation - Croatia and Slovenia as EU member states, while Serbia did it in light of the EU accession process. Despite this fact and the fact that regulations in all three countries were the same and developed in the same manner until 1990s, the variety of fire safety requirements or even some definitions is present.

The structure of regulatory system in all three analysed countries is based on the same principals. The main regulations defining general principles on fire safety are the Law on Fire Protection (Serbia), Fire Protection Act (Croatia) and Fire Protection Act (Slovenia). Detailed requirements are regulated by bylaws, guidelines and standards [7]-[9]. In Croatia, the bylaw is Ordinance on Fire Resistance and other Requirements for Buildings in Case of Fire (hereinafter the Ordinance). Slovenian bylaw Rules on fire safety in buildings, prescribes Technical guideline TSG-1-001:2019 for Fire Safety in Buildings (hereinafter the Technical guideline), as mandatory document for fire safety design [10]. These documents combine the requirements for (among other) residential buildings and fire safety of façades [11], [12]. In Serbian regulatory system, the Regulation on Technical Requirements for Fire Protection of Residential, Business and Public Buildings prescribes fire safety requirements for residential buildings in terms of design approach, performance requirements, etc. Fire safety of façade is regulated by separate document: Regulation on Technical Requirements for Fire Safety of External Building Walls (hereinafter Regulation) [13]. Additionally, while in Serbia and Slovenia fire safety of high-rise buildings is defined in separate documents (Serbia-regulation, Slovenia-guidelines), Croatian regulatory system incorporated these requirements in the Ordinance [11], [13]-[15]. The fire classification of construction products and structural elements is based on EU standard EN 13501-1:2010 Fire classification of construction products and building elements in all three countries, making the basis for comparison of performance requirements. In general, national standards in each country are adopted EU standards, while the regulatory differences reflect in bylaws, adjusted to national specificities.

The base for comparative analysis of fire safety legal framework is derived from each country's mandatory regulations on fire safety of façades, namely *Regulation on Technical Requirements for Fire Safety of External Building Walls* (Serbia, Official Gazette 59/2016, 36/2017 and 6/2019), *Ordinance on Fire Resistance and other Requirements for Buildings in Case of Fi*re (Croatia, Official Gazette 29/13, 87/15) and *Technical guideline TSG-1-001:2019 for Fire Safety in Buildings* (Slovenia) [11]-[13], as well as other supplementary regulations and technical guidelines on fire safety (i.e. criteria for defining the high-rise buildings).

For the purpose of analysing the current state in each country's legal framework, residential buildings façade requirements are compared as an example, while other building purpose groups are excluded. Building design regulations in all three countries use building classification based on European Union Classification of Types of Construction (CC) [16], but additional classifications are used [12], [13], [17], taking into the consideration the relevant building characteristics regarding façade fire safety (e.g. building height and gross floor area). The Table 1 shows building categories listed in regulations and descriptions of residential buildings in each category. The Serbian regulation classifies buildings in 5 categories, with high-rise buildings being left out and regulated with specific regulation, which is not focused only on façades, but all other structure elements as well [15]. The same number of categories is recognized in Croatian Ordinance with addition of the requirements for high-rise building façades, while Slovenian Technical guideline use CC classification and additionally divide buildings by height. Despite the fact that those categories refer to other building purpose groups as well, the Table 1 shows only types of residential buildings, based on the inclusion criteria of this study.

Table 1
Building classification in Serbian, Croatian and Slovenian regulations [11–14]

	Serbia	
Category	**Description**	
A	no residential buildings	
B	Residential, mixed occupancy (residential-business)with gross floor area ≤400m2	
V1	Residential, mixed occupancy (residential-business) buildings with gross floor area 400m2 - 2000m2 and height ≤15m	
V2	Residential, mixed occupancy (residential-business) buildings with gross floor area > 2000m2 and height 15m - 22m.	
G	Residential, mixed occupancy (residential-business) buildings with height 22m-30m	
	Croatia	
ZPS 1	Detached buildings accessible to the fire-fighters from minimum three sides for the purpose of extinguishing fires from the ground level, they have up to three above-ground floors, with the height of the highest residence floor of maximum 7 meters measured from the fire service access level or from where the evacuation of endangered people is possible, and which contain one resident unit of up to 400 m^2 gross floor area and with up to 50 occupants.	
ZPS 2	Detached and semi-detached buildings, with up to three above-ground floors, with a 7 meter height of the residence floor measured from the outside elevation from the fire service access level and from where the evacuation of endangered people is possible, and which contain maximum three resident units of single gross floor area up to 400 m^2 and with a total of up to 100 occupants.	
ZPS 3	Buildings with three above-ground floors with the height of the highest residence floor up to 7 meters measured from the outside elevation from the fire service access level and from where the evacuation of endangered people is possible, where fewer than 300 persons gather. These buildings are not included in subgroups 1 and 2.	
ZPS 4	Buildings with up to four above-ground floors with the height of the highest residence floor up to 11 meters measured from the outside elevation from the fire service access level and from where the evacuation of endangered people is possible, and which include one resident unit without limitation in gross floor area or more resident units of single gross floor area up to 400 m2 and a total of up to 300 occupants.	
ZPS 5	Buildings with the height of the highest residence floor up to 22 m measured from the outside elevation from the fire service access level and from where the evacuation of endangered people is possible, and which do not belong to the subgroups ZPS 1, ZPS 2, ZPS 3 and ZPS 4, as well as buildings which mostly consist of underground stories, buildings where immobile persons or persons with reduced mobility reside	
ZPS 6	High-rise buildings (>22m)	
	Slovenia	
11	Residential buildings	Up to 10m height; from 10m to high-rise (>22m)
111	One-dwelling buildings	
112	Two- and more dwelling buildings	
11301	Residential buildings with service residences for the elderly	
11302	Other residences for communities	

The regulation approach is the same in all three countries. The height of the building is recognized as important factor for determination of fire risk in each regulation, which resulted in building classification according to its height. The height is defined as the distance from ground floor approachable by fire trucks to the highest residence floor level [11], [12], [15]. Slovenian Technical guidelines only refer to buildings up to 10m high and more than 10m to high-rise height, where the high-rise building is defined as building with height >22m [12]. Serbian regulation defines building categories by setting the building height limit at 15m and 30m and Croatian regulation sets this limit to 7m and 22m (see Table 1). This is in line with definition of high-rise buildings in each country (Serbia >30m, Croatia >22m) [11], [15]. Single gross floor area is defined only in Croatian Ordinance, while building gross floor area is defined in Serbian regulation, and along with the building height, describes building categories. Number of residence units and floors are relevant in Slovenian and Croatian regulations. In addition, Croatian Ordinance takes into consideration the number of occupants in building and the position of other buildings (e.g. detached and semi-detached building) [11]-[13], [17]. These kind of classification criteria are present in other Serbian regulations [18], but in regulations on façades are not listed. The findings of building classification analysis showed potential difficulties in conclusive matching the building classes from different regulations, due to the inconsistency in classification criteria.

After defining the types of residential buildings as subjects of analysis, criteria for conducting the analysis had to be defined. In order to meet the general requirements for providing the necessary building fire safety level (e.g. to obtain the load-bearing capacity of the structure during fire, preventing the spread of fire, safe evacuation of people, etc.) a set of requirements for building materials and elements is prescribed in regulations. These include reaction to fire of applied materials, fire resistance of structure elements, horizontal and vertical façade fire barriers, separation of buildings, etc. [7], [8], [19]. The key aspect of each regulation that has the significant impact on fire safety and performance of façades was detected, which resulted in establishing reaction to fire requirements as criteria for comparative analysis [20]. Additional remarks were made regarding the horizontal and vertical façade fire barriers requirements.

2.1. Reaction to fire requirements

Reaction to fire is defined as material response to fire in terms of contribution to fire during the exposure by its own decomposition. In each country's regulations, material response in fire is classified according to the European Reaction to Fire Classification System (Euroclass), defined in European standard EN 13501-1: Fire classification of construction products and building elements, adopted as the national standard. There are seven reaction to fire classes - A1, A2, B, C, D, E and F describing material's combustibility, where A1 and A2 classes stand for non-combustible materials. Additionally, three smoke production classes are assigned as criteria for reaction to fire - s1, s2 and s3, where s1 stands for small amount of smoke production during fire to s3 substantial smoke production. Furthermore, criteria on flaming droplets and/or particles defines 3 classes: d0, if no flaming droplets/particles occur within 600s; d1, if no flaming droplets/particles, persisting longer than 10s, occur within 600s; d2, if no performance is declared or if the product does not comply with d0 or d1 classification criteria [21]. Detailed reaction to fire requirements for each country depending on building categories and façade types are given in Table 2, Table 3 and Table 4.

Table 2
Reaction to fire requirements for façades- Serbian regulations [13]

Type of façade	Building category			
	B	**V1**	**V2**	**G**
Masonry (bricks, blocks, etc.) and concrete (precast or cast-in-place) walls with thermal insulation and with external masonry, concrete or similar cladding for protection from weathering - non ventilated; pre-casted self bearing façade panels				
External wall system	C-s2, d2	B-s1, d1	A2-s1, d1	A2-s1, d1
Components of external wall				
External layer(s)	B-s2, d1	B-s2, d1	A2-s1, d1	A2-s1, d0
Thermal insulation	E-s2, d2	E-s2, d1	D-s2, d1	C-s2, d1
Masonry (bricks, blocks, etc.) and concrete (precast or cast-in-place) walls with thermal insulation and with external masonry, concrete or similar cladding for protection from weathering - ventilated				
External wall system	C-s2, d2	B-s2, d1	A2-s1, d1	A2-s1, d0
Components of external wall:				
External layer(s)	C-s2, d2	B-s2, d1	A2-s1, d1	A2-s1, d0
Substructure				
Dowel type substructure	C	B	A2	A2
Dotted substructure	A2	A2	A2	A2
Thermal insulation	B-s2, d1	A2-s1, d0	A2-s1, d0	A2-s1, d0
Masonry (bricks, blocks, etc.) and concrete (precast or cast-in-place) walls with ETICS				
External wall system	D-s2, d2	B-s2, d1	B-s1, d1	A2-s1, d0
Components of external wall:				
Finishing layer(s)	C-s2, d1	B-s2, d1	B-s1, d1	A2-s1,d0
Thermal insulation	E-s2, d2	B-s2, d1	A2-s1, d1	A2-s1, d0

Table 3
Reaction to fire requirements for façades - Croatia [11], [17]

Construction parts	Subgroup of buildings					
	ZPS1	**ZPS2**	**ZPS3**	**ZPS4**	**ZPS5**	**High-rise**
Suspended ventilated elements of façades						
Classified system	E	D-d1	D-d1	C-d1	B-d1	A2-d1
Execution with the following classified elements:						
Finishing layer	E	D	D	A2-d1 or B-d1	B-d1	A2-d1
Substructure						
Dowel type substructure	E	D	D	D D	C	A2
Dotted substructure	E	D	A2	A2 or A2	A2	A2
Insulation	E	D	D	B A2	A2	A2
Thermal contact system of façades (ETICS)						
Classified system	E	D	D-d1	C-d1	B-d1	A2-d1
Composition of layers with the following classified components:						
Finishing layer	E	D	D	C	B-d1	A2-d1
Insulation layer	E	D	C	B	A2	A2

Table 4
Reaction to fire requirements for façades – Slovenia [12]

Groups of (residential) buildings	**Building height**	
	up to 10m	**from 10 to high-rise buildings**
External wall claddings		
One dwelling buildings	D-s3,d2	B-d0
Two-and more dwelling buildings	D-s3,d2	B-d0
Residential buildings with service residences for the elderly	ground-floor buildings D-d0, buildings with several above-ground floors B-d0	A1 or A2
Other residences for communities	for ground-floor buildings D-d0, for buildings with several above-ground floors B-d0	A1 or A2
Sandwich panels with metal skin on both sides		
Residential buildings	A2-s1, d0	
Composite systems for external thermal insulation (ETICS) with combustible insulation		
not specified	B-d1	A1 or A2
`Ventilated façades (insulation)		
not specified	A1 or A2-s1, d0	A1 or A2-s1, d0

Types of façades in Serbian and Croatian regulations are defined in the same way, considering the possibility of their use as a system or as separate classified components. While Serbian Regulation refers to non-ventilated, ventilated and ETICS (External thermal insulation composite system) façade systems, Croatian Ordinance prescribes the set of requirements for ventilated and ETICS façade systems. Although the building categories could be compared since at least one classification criteria is overlapping, different value intervals of compared characteristics leads to non-unified categorization. Therefore, the comparison is not completely transparent. Façade types in Slovenian regulation are not defined as a system and components like in Serbian and Croatian regulations, which results in inability to compare the requirements in a precise and exact way.

The use of F Euroclass materials is not allowed in any of referenced regulations. The lowest material class that can be applied is E in Serbia and Croatia and D in Slovenia. The smoke spread and the flaming droplets/particles requirements are defined in Serbian regulation for all types of façade systems and components (only d0 and d1 classes are allowed), except substructure elements. Smoke production requirements are not prescribed in Croatian regulations, and for some of the façade elements only the combustibility class is defined (see Table 4). Non-combustible materials are mandatory for thermal insulation of ventilated façades in Slovenian Technical guidance, regardless the building class and height. This criteria is prescribed for V1, V2 and G classes in Serbian Regulation, and ZPS 4, 5, 6 in Croatian. Furthermore, non-combustibility requirements are defined for: building height from 15m in Serbian regulation (V2 and G building category, except thermal insulation of non-ventilated façades and ETICS, when installed as external wall system), in Croatian for some elements even for buildings with 7m height (e.g. dotted substructure) (categories ZPS3, ZPS4, ZPS5 and high rise buildings) and in Slovenian mainly for the buildings higher than 10m with the exception of sandwich panels (non-combustible materials are applied in buildings with height lower than 10m).

Considering listed differences, reaction to fire requirements in Serbian, Croatian and Slovenian regulations can be compared with certain adjustments of building classifications. The consequences of differences in building classification and reaction to fire requirements as well as mutual impact of these two criteria is explained in following example. In Table 5 the characteristics of three buildings are listed. The buildings are classified according to the criteria in each country's regulation. Furthermore, for each building class, the reaction to fire requirements for ETICS façade system are defined.

Table 5
Differences in building classifications and reaction to fire requirements in Serbia, Croatia and Slovenia - example

<table>
<tr><td>Building description
Country</td><td colspan="2">Building 1:
• H=9.5m
• four above-ground floors,
• 4 residential units on each floor,
• 64 occupants,
• gross floor area: 250m2
• gross area: 1000m2</td><td colspan="2">Building 2:
• H=12.5m,
• five above-ground floors,
• 4 residential units on each floor,
• 80 occupants
• gross floor area: 250m2
• gross area: 1250 m2</td><td colspan="2">Building 3:
• H=25m</td></tr>
<tr><td rowspan="2">Serbia</td><td rowspan="2">V1</td><td>system: B-s2, d1</td><td rowspan="2">V1</td><td>system: B-s2, d1</td><td rowspan="2">G</td><td>system: A2-s1, d0</td></tr>
<tr><td>finishing layers: B-s2, d1
insulation: B-s2, d1</td><td>finishing layers: B-s2, d1
insulation: B-s2, d1</td><td>finishing layers: A2-s1, d0
insulation: A2-s1, d0</td></tr>
<tr><td rowspan="2">Croatia</td><td rowspan="2">ZPS4</td><td>system: C</td><td rowspan="2">ZPS5</td><td>system: B-d1</td><td rowspan="2">ZPS6</td><td>system: A2-d1</td></tr>
<tr><td>finishing layers: C
insulation: B</td><td>finishing layers: B-d1
insulation: A2</td><td>finishing layers: A2-d1
insulation: A2-d1</td></tr>
<tr><td rowspan="2">Slovenia</td><td rowspan="2">112</td><td>system: B-d1</td><td rowspan="2">112</td><td>system: A1, A2</td><td rowspan="2">high-rise</td><td>system: A1, A2</td></tr>
<tr><td>NA</td><td>NA</td><td>NA</td></tr>
</table>

Table 5 shows that level of matching between reactions to fire criteria mostly depends on the building height. For the building 1, Serbian and Slovenian regulations prescribe B fire class for façade systems, although smoke production class is defined only in Serbian regulations (see Table 5). Croatian Ordinance requirements are less strict, allowing C-d1 class for façade systems. Regarding the façade components, Serbian regulation prescribes B class for both finishing and insulation layer, while C class is allowed for finishing layer and B class for insulation by Croatian Ordinance. Since building 2 is higher than building 1, it is classified differently in Croatian and Slovenian regulations due to additional criteria defined, while it keeps the same class in Serbian regulations. This resulted in reaction to fire requirement being the same for the façade system in Serbia and Croatia (class B), while Slovenian regulations require A1 or A2 class. Regarding the façade components, Croatian regulation is stricter for the thermal insulation layer, prescribing A2 combustibility class. Finally, the complete matching is possible for building 3 (i.e. building height >22m), where all three regulations require the combustibility class A1 or A2 (non-combustible materials), with exception of Serbian regulation which prescribes both smoke production class and flaming droplets/particles class as well. This points out the variety of requirements, more in strictness of prescribed reaction to fire classes (due to the different building classification and differences in maximum height within one class), than in the general fire safety concept.

2.2. Horizontal and vertical fire barriers - general notes

Vertical and horizontal fire barriers on façades are used for preventing the fire spread on façades [11]-[13], [20]. Minimum requirements for horizontal and vertical barriers vary in dimensions, while the concept is the same for each country - the openings are rounded with non-combustible materials and horizontal continuous belts are built in the façade of alternating floors for preventing the vertical fire spread. The width of the bands must be at least equal to the width of thermal insulation in façade system. Horizontal fire spread is prevented by built in non-combustible vertical barriers.

In order to prevent the fire spread from one fire compartment to another, Serbian Regulation on Technical Requirements for Fire Safety of External Building Walls prescribes the designing of horizontal and vertical fire barriers if the façade is not entirely made from non-combustible materials. The barriers must be built in along the borders of the fire compartments and along the junctions of internal fire walls and external wall. The schematic representation of this concept is shown in Figure 1 and Figure 2. Reaction to fire class of barrier is defined depending on its position. Class A1 is mandatory along the borders of the fire segments (part of the building consisting of two or more fire compartments) and A2 on the borders of the fire compartments.

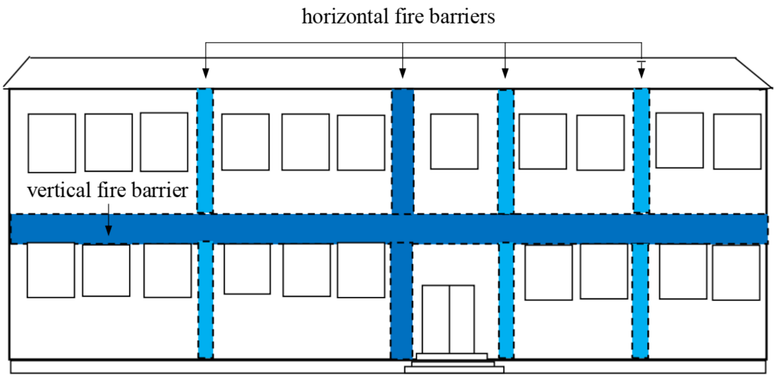

Figure 1. Horizontal and vertical fire barriers along the borders of fire compartment (dark blue) and the junctions of internal fire walls and external building wall (light blue) [13].

Furthermore, the set of minimum requirements for barrier's dimensions are regulated. Minimum height (h_p) and width (w_p) of the vertical and horizontal fire barrier is 1m. Alternatively, these dimensions can be reduced if the non-combustible console element and bracket are installed, with the minimum width p>0.5 m (Figure 3). Even more strict requirements must be fulfilled when reconstructing the façade of existing building - if the ETICS façade system installed is not entirely made of non-combustible material, they must have vertical fire barriers along the contact with floor construction of each alternating floor.

The minimum height of these barriers is 1m, and the reaction to fire class must be A1, or in accordance with the requirements for a higher building (see Table 2) [13].

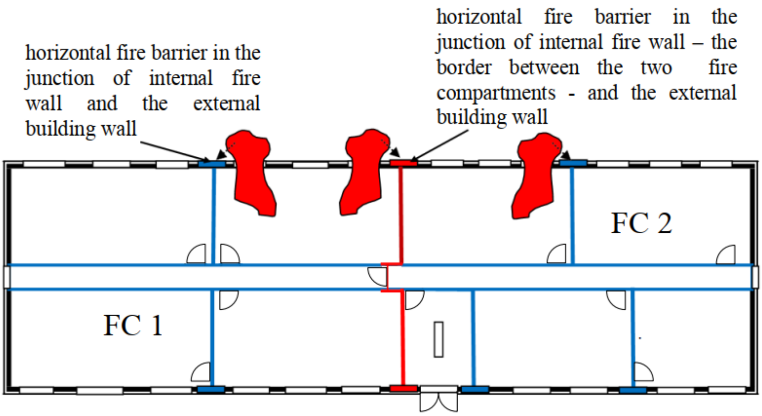

Figure 2. Vertical fire barriers along junction of the border of two fire compartments and external building wall (red) and along junctions of the internal fire walls and external building wall (blue) [13].

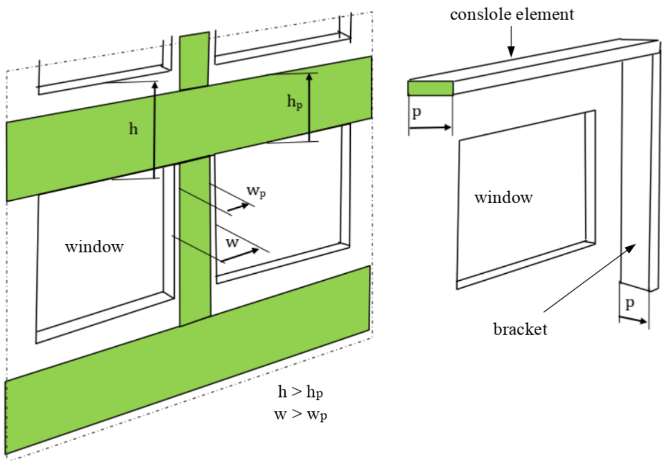

Figure 3. Horizontal and vertical fire barriers on façade - continuous bands (on the left) and continuous console element supported by the vertical bracket (on the right) [13].

The protection of combustible roofing or eave from vertical fire spread must be provided by building into the façade an A1 reaction to fire class horizontal band above the openings at the highest floor, with the minimum height of 1m, extended for at least 0.5m from the both sides of the openings. This is not mandatory if the combustible roofing material or eave is covered with non-combustible lining.

Croatian Ordinance prescribes A1 or A2-s1, d0 classes for barriers preventing horizontal and vertical fire spread. Fire barriers must be made on fire resistant construction elements that prevent horizontal and vertical fire spread (e.g. fire walls and fire resistant parapets between the openings of different fire compartments) when making ETICS façade systems with combustible thermal insulation [11]. The illustrations of requirements for horizontal fire barriers - their position and minimum dimensions depending on the building category (see Table 1) - are shown in Figure 4. Another provision on preventing the horizontal fire spread refers to corner joints of two fire compartments. Prescribed minimum width is 3m or 5m, depending on building category, as shown in Figure 5. Similar to the Serbian regulation, it is required by the Croatian Ordinance to protect the roof of a building from fire spread. It is mandatory to finish the fire wall with a non-combustible insulation material (Figure 6).

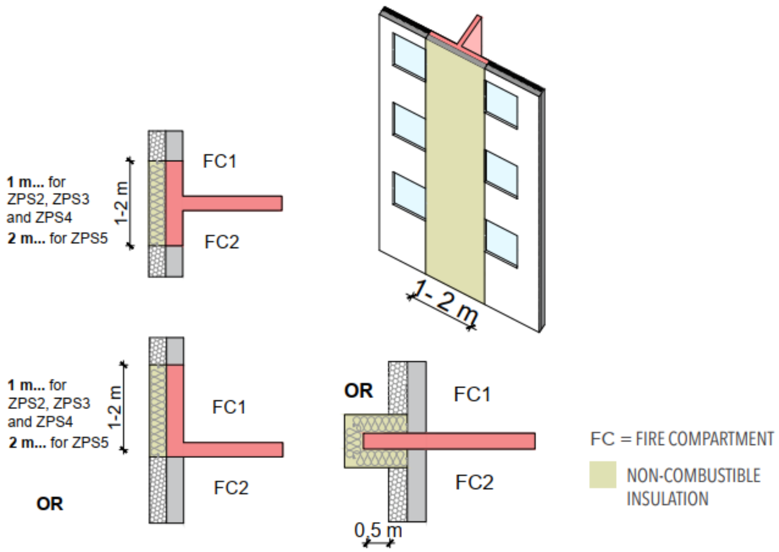

Figure 4. The example of horizontal fire spread prevention with vertical non-combustible insulation across the fire wall [17].

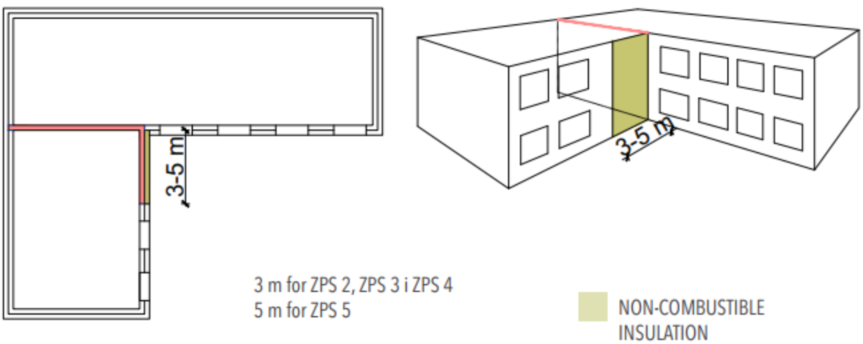

Figure 5. Horizontal fire barrier on the corner joint of two fire compartments [17].

Vertical fire spread between two floors that are different fire compartments is prevented by making horizontal fire barriers between the openings. The minimum height of the barriers is shown on the Figure 7 [11], [17].

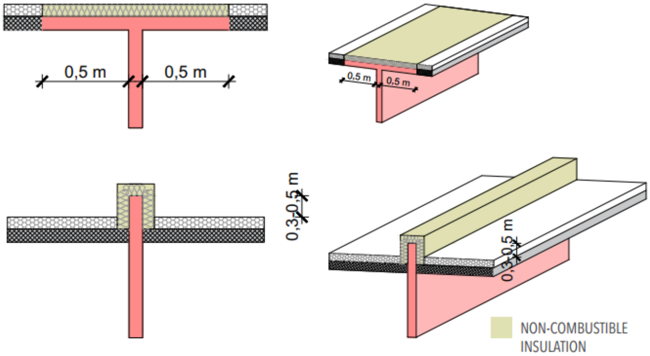

Figure 6. Roof protection - non-combustible material around the finishing of a fire wall [17].

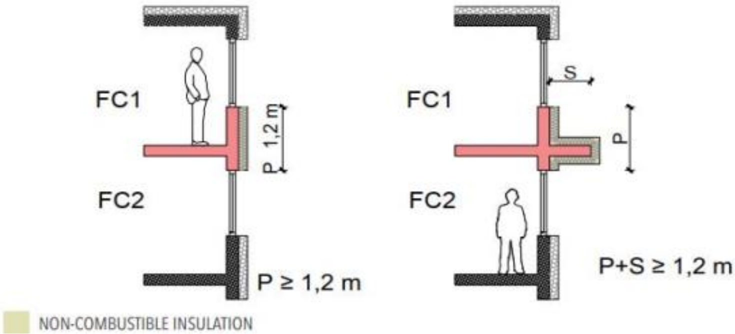

Figure 7. Requirements for horizontal fire barriers [17].

To prevent the fire spread across the façade with combustible thermal insulation within one fire compartment, for the building category ZP4, fire barriers are built in around the openings. (Figure 8). Alternatively, horizontal continuous bands can be made round the building, on each floor, in a way illustrated in Figure 9.

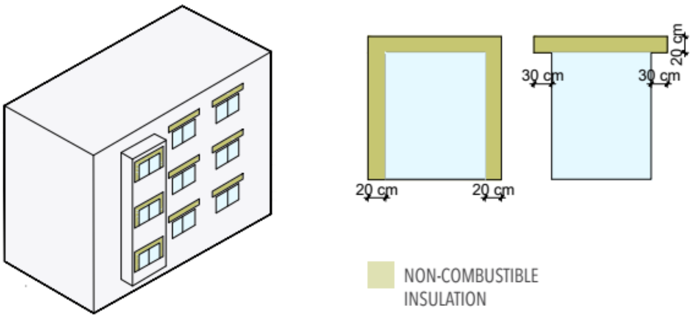

Figure 8. Fire barriers around the openings - fire spread prevention within the fire compartment [17].

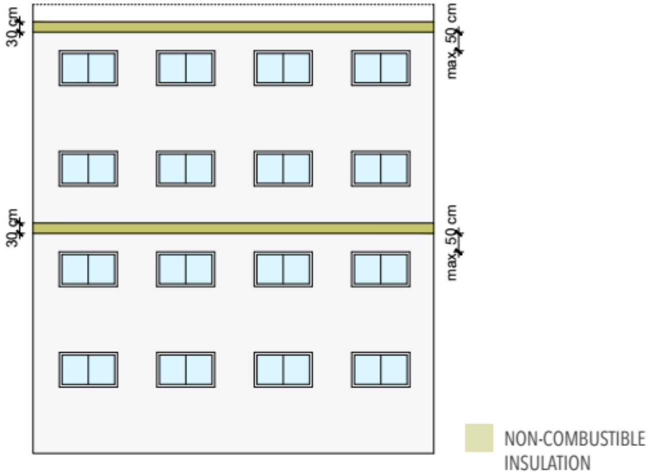

Figure 9. Prevention of the vertical fire spread by making a vertical fire barrier along the junction of the floor construction of each floor and external wall [17].

Reaction to fire of barriers is defined as at least A2-s1, d0 in Slovenian technical guidance. Basic requirements for preventing the fire spread consist of providing a sufficient distances - barriers - between windows, covered with non-combustible materials. The internal corners of joint fire compartments must be protected with horizontal fire barriers, if the angle between external walls of fire compartments is 135 degrees or less (Figure 10). The distance between these walls, or the width of the barrier, depends on the expected fire load. Also, the presence of the extinguishing system on the façade is recognized as changing factor. For the buildings that have them installed, the minimum width of horizontal fire barrier can be reduced.

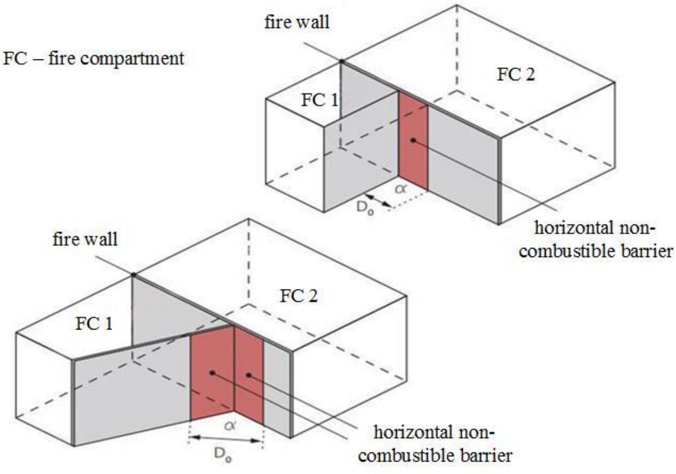

Figure 10. Prevention of horizontal fire spread between two fire compartments [12].

Vertical fire spread across the façade must be prevented by horizontal fire barriers placed across the fire resistant parapets of the floor inaccessible by fire brigade. The minimum requirements in terms of dimensions is show in Figure 11. Again, if a sprinkler system is installed in the bottom and the top of fire compartments, it is not obligatory to design the mentioned barriers [12].

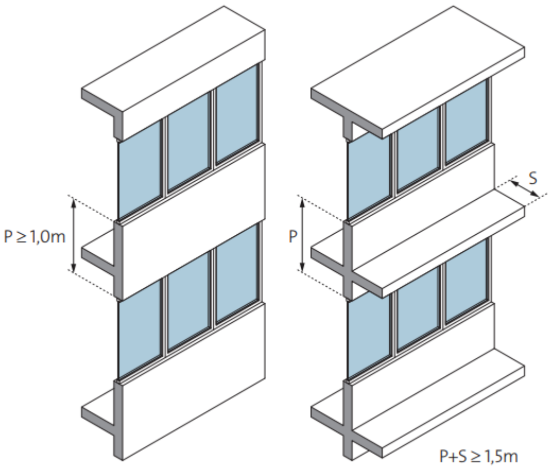

Figure 11. Prevention of the vertical fire spread - minimum dimensions of fire barriers placed across the fire resistant elements [12].

Regulations in each country also set the requirement for designing a horizontal fire barrier for preventing the vertical fire spread through ventilated layer of ventilated façades with combustible or non-combustible thermal insulation [11]-[13].

3. CONCLUSIONS

By reviewing the current regulations in Serbia, Croatia and Slovenia, it is concluded that the general framework and approach to fire safety is based on the same general principles in all three countries. Each country has regulations adjusted to national specificities, while national standards are adopted EU standards.

The regulations where analysed by two façade performance-related criteria: reaction to fire and general requirements for horizontal and vertical fire barriers. Furthermore, differences in building classifications are recognized as the main issue in applying the regulations in practice, since it is the first step in defining the performance requirements. Although building classes from one regulation could be matched with the classes from other with certain adjustments, there is no possibility to establish the conclusive system, since the criteria for classification vary in each country's regulation. Reaction to fire requirements vary in terms of the level of requirements, depending mostly on height of the assessed building. While Serbian and Slovenian regulations prescribe the higher requirements for lower buildings, this changes with the increase of building height, where Croatian and Slovenian regulations become more strict than Serbian. Complete match in reaction to fire requirements is possible only for buildings higher than 22m. Additionally, regulations refer to different façade descriptions and types. This makes the comparison and application of different regulations non-unified and less exact. By analysing the minimum requirements for horizontal and vertical barriers in all three countries regulations, it is concluded that the concept of designing the barriers is the same. Insignificant variations occur in dimensions, but in all regulations the use of non-combustible materials for horizontal and vertical barriers is mandatory.

The study presented in this book chapter should be considered as an initial step towards the comparison of Serbian, Croatian and Slovenian regulations, while, for more detailed analysis, additional studies should be conducted, considering all fire safety requirements. These kinds of analysis are deemed as the basis for creating the single market in Serbia, Croatia and Slovenia with the support of implemented CPR as a tool. Although CPR prescribes harmonized rules for the construction products market, the regulations in these countries should provide the same concepts and level of fire safety requirements. In order to support the single market, a certain level of regulation transparency in interpretation should be provided as well, so the regulations could be understood and implemented more easily in design and construction practice in the region.

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APPENDIX

FAÇADE FIRE SAFETY LEGAL FRAMEWORK IN ALBANIA

Sokol Dervishi

1. INTRODUCTION

Legal framework of Albania consists of two decisions:

- Minister of Internal Affair's Order No. 424, dated 24.7.2015: On the approval of technical rules for fire protection and rescue in buildings intended for housing;
- Council of Minister's Decision No. 626, dated 15.7.2015 "On the approval of housing design norms".

2. GENERAL PRINCIPLES OF THE LEGAL FRAMEWORK

Fire Safety Design of buildings is based on understanding fire resources and materials and systems that would fire and spread fire. Recommendations and instructions provided in this legal framework are based on the principle that in normal circumstances it is unlikely that fire could start in two difference places in a building. Recommendations given in this standard are of general nature and for all measures and procedures on fire safety and protection. Special circumstances of each building or complex are taken into account. The same recommendations are generally applied for both the existing and new buildings. However, the existing buildings, especially the historical buildings, are often faced with problems which the new buildings are unlikely to face. This standard provides recommendations and instruction on design, management and use of buildings, in order to satisfy reasonable standards on fire safety, for all individuals who are in the building. This framework does not include information for one family dwelling. The legal framework is structured to be applied for the design of new buildings and different renovations and retrofitting measures of the building. Furthermore, it provides instructions on the continuous management of fire safety during the entire cycle of duration of the building, including instructions to designers, in order to provide that the general project of a building helps to improve fire safety management.

3. RESISTANCE OF STRUCTURES TO FIRE

Fire resistance requirements of construction structures is evaluated on the basis of descriptions and test modalities, specified by special provisions, as well as bylaws, depending on the type of material used (e.g. concrete, steel, solid wood, laminated wood, composite elements, varnished materials). For specific fire risk structures, the relevant norms are to be applied by separate bylaws.

4. CONSTRUCTIVE CHARACTERISTICS

Table 6
Classification of the housing typologies with associated REI characteristics

Housing typology	Maximum Surface (m2) of the building	Maximum Surface (m2) of floor (according to the housing category)	Stair cage typology (at least one cage for the elevator)	"REI" characteristics of the walls (of the stair cage, elevator, smoke filter, emergency doors, subdivision elements on the floor)
a	From 12m to 24 m	8000	500	60 (**)
			500	60
			550	60
			600	60
b	From 24 m to 32 m	6000	500	60 (**)
			550	60
			550	60
			600	60
c	From 32 m to 54	5000	500	90
d	From 54 m to 80	4000	500	90
e	Above 80 m	2000	350 (*)	120

(*) With a minimum of 2 stairs for each floor compartment. In the terrace of the building should be provided a place for landing and the setting up of first aid helicopters, which is accessible by any degree.
(**) Only for subdivision elements between room.
Fire resistance of separation structures and systems shall respectively guarantee REI requirements, as specified in the table 2: For structures and systems with specific risk zones, including gates and special dividing and locking elements, the relevant norms provided for in the by-laws shall apply.

Table 7
Classification of the housing typologies with associated REI characteristics

Floor	REI
intermediate floors	REI 60-90;
buildings in anti-fire height to 24 m	REI 90
buildings above 24 m	REI 90-120

5. CONCLUSIONS

The legal framework includes the risk analysis and evaluation, the principles of the safety measures (e.g. detection system and fire alarm, escape signs, doors and elevator) and placement of evacuation vertical systems (stairs and elevators), evacuation capacity, firefighting active systems.

However, the frameworks lack considerably the information and guidelines regarding the fire safety of the façades, including the codes and the materials used with respect to fire safety standards. As such, a detailed investigation needs to be explored to improve the legal framework.

ENERGY REHABILITATION AND FAÇADE FIRE SAFETY OF HIGH-RISE RESIDENTIAL BUILDINGS

Suzana Draganić, Mirjana Malešev, Olivera Bukvić, Mirjana Laban

1. INTRODUCTION

Buildings are responsible for approximately 40% of EU energy consumption and 36% of the CO_2 emissions, making them the single largest energy consumer in Europe. Currently, about 35% of EU's buildings are over 50 years old and almost 75% of the building stock is energy inefficient, while only 0.4-1.2% of the building stock is renovated each year [1]. Consequently, potential energy savings in building sector are greater than in any other sector, while improving the energy performance of buildings is one of the crucial factors for reducing greenhouse gas emissions.

Energy rehabilitation of façade walls is one of the key measures for improving the energy performance of existing buildings. At the same time, the installation of an additional thermal insulation layer on facades is also an integrative factor in the renewal and improvement of the quality and the safety of the building and its occupants [2].

Previous research [3] on the possibilities of improving energy efficiency in buildings has identified the need for energy rehabilitation of facades, which can reduce heating energy losses by up to 60%, as well as the need for improving residential comfort, as well as building fire safety.

Safety in case of fire and *energy economy and heat retention* represent two out of seven basic requirements that construction products and buildings must satisfy for an economically reasonable working life, subject to normal maintenance, which applies also to the existing buildings in renewal process [4], [5]. As the issues of façade energy rehabilitation are closely related to fire safety issues, applied solutions for renewal have to meet both the energy efficiency, as well as fire safety requirements.

The basic fire safety requirement is satisfied if in the event of fire [4]-[6]:

- the load-bearing capacity of the structure is maintained for a specific period of time;
- the generation and spread of fire and smoke within the building are limited;
- the spread of fire to neighbouring buildings is limited;
- occupants can leave the building or be rescued by other means;
- the safety of rescue teams is taken into consideration.

The design and materialization of façade walls significantly influence the requirements relating to the generation and spread of fire and smoke within the building and the spread of fire to neighbouring buildings. Therefore, within the façade energy rehabilitation, selection of materials with satisfactory thermal but poor reaction to fire performance can contribute to the spread of fire along the facade and can result with catastrophic consequences – as evidenced by the number of façade fires in the world.

In the Republic of Serbia there are 3.23 million of dwellings [7] which clearly indicates the potential for energy savings in this sector. Housing stock built in the second half of the XX century is built according to the currently out-dated energy regulations which results with the average energy consumption that exceeds 150 KWh/m^2 per year, while in developed European countries is below 50 kWh/m^2 [8]. In addition, this period characterize the modest fire safety regulations, that did not provide detailed guidance, so today these buildings pose a particular problem when it comes to taking protective measures to reduce the risk of fire and its spread when a fire occurs. In order to improve the current situation, it is necessary that Serbia accelerate activities related to the achievement of EU standards.

In the second half of the XX century more than 41.223 apartments were built in Novi Sad. Most of the buildings are built with industrial building technology (50%), while others are built in a traditional manner (36%) or by applying advanced construction technology (14%) [9]. More than 70 buildings are freestanding high-rise residential buildings (buildings with ground floor and at least 10 floors), mostly built with the prestressed industrial prefabricated IMS building technology. Previous research [10] has shown that the fire safety level of these buildings is very low. The collected and processed statistics on the fire occurrence indicate a constant increase in the number of fires in multi-storey residential buildings, as well as the increase in the percentage of residential fires in the total number of fires [11]. After years of exploitation, unsatisfactory technical condition of most façade elements and lack of regular maintenance, there is a need for renewal of these buildings through improvement of façade performances, in order to comply with the requirements of contemporary technical regulations and standards.

2. FAÇADE FIRE SAFETY OF HIGH-RISE BUILDINGS

Façade fire safety in high-rise buildings in the Republic of Serbia is regulated by the *Law on Fire Protection* [6], as well as the *Regulation on technical norms for the protection of high-rise buildings from fire* [12]. Meeting the fire safety requirements of the building proves with the *Elaborate on fire safety* that has to be enclosed with the technical documentation in order to obtain approval for execution of energy rehabilitation [13].

During the second half of the XX century the Law [6] was revised and modified several times [14], [15].

The first regulation relating exclusively to high-rise buildings [16] was adopted in 1984. High-rise building was defined as *a building where a height of its last inhabited storey's floor exceeds 22 m in relation to the surrounding ground that is accessible to the fire-fighting and rescue vehicles and from which it is possible to intervene with the use of motor ladders or other special vehicles for extinguishing and rescuing from heights*. The materials used for construction of supporting and reinforced façade walls had to be non-combustible and fire resistant at least 1.5 h. It was allowed to install combustible insulating material on the external side of the walls provided that it was properly adhered to the walls and anchored so that it does not fall off in the event of fire and that it was coated with non-combustible material resistant to fire for at least 1 h. If the combustible insulating material was installed within the wall layers, the cladding on the inside had to be fire resistant for at least 1.5 h. For application on internal side of the walls, the insulation material had to be non-combustible. In addition, the external building walls had to be constructed of materials that would limit fire spread from one floor to another, with at least 1 m high vertical fire barriers between openings on two adjacent floors. If this distance was less than 1 m, the flame path between the two floors was extended by placing the cantilever slabs at the level of each floor structure.

The height of high-rise buildings increased to 30 m by modification of the Regulation [16] in 2011, which did not affect the requirements for façade walls. In 2015, the

Regulation was replaced by a new one that has been modified twice in the following period [12].

The current Regulation [12] prescribes the technical requirements for fire safety that must be fulfilled when designing, constructing, reconstructing and upgrading the high-rise buildings. In case of building reconstruction, the defined requirements are only applicable to the part of the building subjected to reconstruction and the fire safety level of the existing building must not be reduced by reconstruction.

External building load-bearing walls have to be fire resistant 2.0 h (RE-M 120) and built from non-combustible building materials with at least A2-s1,d0 reaction to fire class, according to SRPS EN 13501-1 standard [17]. Reaction to fire classification of building materials in façade walls composition is compliant with the European fire classification of construction products.

The fire spread between two adjacent floors along is limited by the construction of a vertical building element with 1.5 h (EI 90) fire resistance. The vertical building element separating the floors (fire barrier) must be at least 1 m high or at least 1.4 m long when the flame path between the two floors is extended by placing the cantilever slabs at the level of the floor structure (Figure 1).

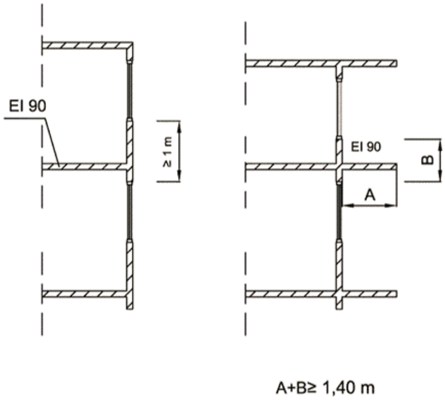

Figure 1. Vertical fire barriers [12]

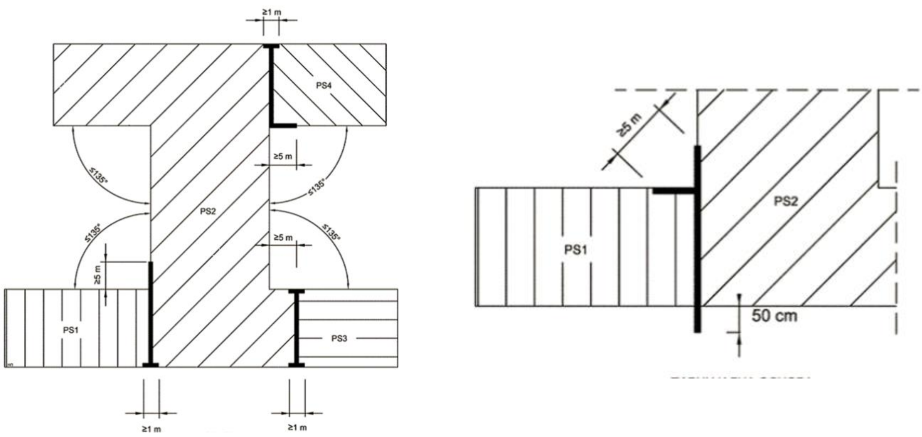

Figure 2. Horizontal fire barriers [12]

Horizontal fire spread along façade at the border of the fire sector is prevented by horizontal fire barriers (Figure 2) by constructing a part of the façade wall in a total width of at least 1 m at the point of the walls' contact. The constructed façade wall has to have the same fire resistance as internal fire wall. Additional way for construction of horizontal fire barrier is to extend internal fire wall beyond the facade plane for 0.5 m. For complex structures where the fire sectors are joined at an angle equal to 135° or less, in order to prevent the horizontal spread of fire from one fire sector to another, a 5 m long wall of the same fire resistance as the wall at the border of the fire sector is constructed in the corner.

In the case of adjacent buildings with different height (Figure 3), on the higher wall there must be no openings at least 10 m above the highest point of the lower building if:

- lower building has openings less than 8 m away from the façade wall of the higher part, or
- floor or roof structure of the lower part, including the roof covering, is not resistant to internal fire for at least 2 hours.

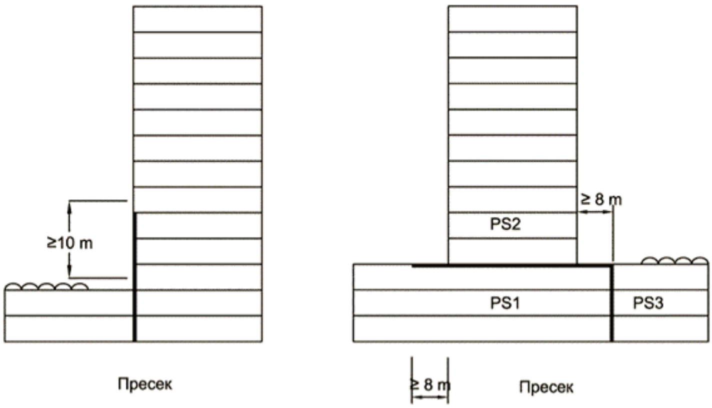

Figure 3. Adjacent buildings with different height [12]

The boundary wall must have a fire resistance of 2 h (REI-M 120) and must be built from non-combustible building materials of at least A2-s1,d0 fire class.

Reaction to fire requirements for façade walls of high-rise residential buildings were also modified over time and evolution of requirements is presented in Table 1.

Table 1
Evolution of reaction to fire requirements for façade walls of high-rise (HR) residential buildings [12]

EXTERNAL WALLS	**2015**	**2017**		**2018 (current)**
	all HR buildings	30 m - 40 m	40 m +	all HR buildings
Wall system	A1	A2-s1,d0	A1	**A2-s1,d0**
Wall components				
External layer(s)	A1	A2-s1,d0	A1	**A2-s1,d0**
Substructure	not specified	not specified	not specified	**A2**
Thermal insulation layer	A1	A1	A1	**A1**

As it can be seen from the Table 1, in case of design, construction and reconstruction of high-rise buildings, all external wall components are required to be non-combustible. The reaction to fire requirements for external wall system have been slightly reduced from 2015 (from A1 to A2-s1,d0 class). The 2017 modification introduced different requirements for two categories of high-rise buildings. However within the latest modification this categorization was excluded and additional requirements for substructure elements have been defined.

The need to improve fire safety in existing and new buildings, as well as the obligation to harmonize national technical regulations with EU regulations, initiated the preparation of *Regulation on technical requirements for fire safety of external building walls* [18]. Regulation prescribes the technical requirements for fire safety of building materials intended for external wall construction which must be fulfilled when designing, constructing, reconstructing, upgrading, adapting, using and maintaining buildings. Buildings are classified into five categories – A, B, V1, V2 and G, aligned to the classification given in [19]. Since the entry into force in 2016, the requirements has been revised and innovated twice. High-rise buildings were primarily (2016) classified into G category (buildings higher than 22 m) with requirement that the wall system and all its components has to be non-combustible (Table 2).

Table 2
Evolution of reaction to fire requirements for external high-rise building walls [18]

Building category	**G** (2016)	**G** (2017)
Pre-casted self-bearing façade panels and masonry (bricks, blocks, etc.) and concrete (precast or cast in-situ) walls with thermal insulation and with external masonry, concrete or similar cladding for protection from weathering - non ventilated		
Wall system	A1	A2-s1,d1
Wall components		
External layer(s)	A2-s1,d1	A2-s1,d0
Thermal insulation	A2-s1,d1	B-s2,d1
Masonry (bricks, blocks, etc.) and concrete (precast or cast in-situ) walls with thermal insulation and with external masonry, concrete or similar cladding for protection from weathering - ventilated		
Wall system	A1	A2-s1,d0
Wall components		
External layer(s)	A1	A2-s1,d0
Substructure		
-dowel type substructure	A1	A2
-dotted substructure	A2	A2
Thermal insulation	A1	A1
Masonry (bricks, blocks, etc.) and concrete (precast or cast in-situ) walls with ETICS		
Wall system	A1	A2-s1,d0
Wall components		
External layer(s)	A2-s1,d1	A2-s1,d0
Thermal insulation	A1	A1

The first modification has led to a reduction in the criteria for the entire wall system, as well as criteria for system components where in some cases the installation of combustible components was allowed (it was allowed to use B-s2,d1 class combustible material for sandwich walls insulation). Within the last modification, G category was limited to 30m in terms of building height which has led to the exclusion of high-rise buildings from the Regulation.

3. ENERGY EFFICIENCY AND THERMAL PROTECTION OF BUILDINGS

The implementation of European legislation in the field of energy efficiency of buildings in Serbia is provided through *Regulation on energy efficiency of buildings* [20] and *Regulation on conditions, content and the way of issuing certificate of energy performances of buildings* [21] effective from 2012. For existing buildings, heat transfer coefficient for external walls is limited to 0.4 W/m^{2}K and the building is energy efficient if the minimum requirements of thermal comfort are fulfilled and if annual energy consumption for heating does not exceed 70 kWh/m^2 per year.

Meeting the energy efficiency requirements of the building is proved with *Elaborate on energy efficiency* that has to be enclosed with the technical documentation in order to obtain approval for execution of energy rehabilitation [13].
Table 3 shows the evolution of the heat transfer coefficient boundary value for façade walls.

Table 3
The evolution of the heat transfer coefficients boundary value for façade walls [20], [22], [24]-[28]

Heat transfer coefficient (W/m^{2}K)	Year of regulation					
	1967	**1970**	**1980**	**1987**	**1988**	**2011**
I climate zone	1,79	1,69	1,225	1,20	1,10	existing buildings: 0,40 new buildings: 0,30
II climate zone	1,55	1,45	0,93	0,90	0,90	
III climate zone	1,37	1,10	0,83	0,80	0,80	

The first requirements for thermal protection of the façade walls were defined in 1967 through the *Regulation on minimum technical requirements for the construction of apartments* [22]. The boundary values of the heat transfer coefficients for the perimeters walls were defined, for three climate zones. Until then, the heat transfer coefficient was not a limiting parameter for the façade walls, as the goal of thermal calculations (carried out according to DIN 4701/1947 standard [23]) was to meet the thermal comfort requirements.

In 1970 the first normative that exclusively concerned the thermal protection of buildings was adopted [24]. The thermal protection requirements have become stricter and the relevant data for calculation were given. It was stated that *the structures and building elements have to be protected against moisture,* which indicated the introduction of the water vapour diffusion parameter in the thermal calculation.

In 1980 the U.J5 mandatory standards for the heat in civil engineering were adopted, which represented a significant step forward in the field of thermal protection of buildings. Standards defined thermal protection requirements in design and construction of buildings [25], heat transfer coefficient calculation methods [26], water vapour diffusion calculation methods [27] and thermal stability calculation methods for external building structures in summer period [28]. In addition to the improvement of thermal protection requirements, the water vapour diffusion and summer regime treatment parameters were introduced in the thermal calculation.

In 1987 thermal protection regulations were modified by introducing the category of specific heat losses for the building as a whole and improving the boundary heat transfer coefficients values. Minor revisions and modifications of standards were made in the following period, while the latest innovation introduced the category of specific heat losses for characteristic rooms.

4. CASE STUDY

Three freestanding high-rise residential buildings S9, S10 and S11 (Figure 4) are located in Novi Sad, in Novo Naselje city area. The thirteen storey buildings are designed by architect Miodrag Milidragovic and built in the 80's. Each building contains 75 apartments.

The buildings are constructed in a pre-fabricated IMS system with a 4.80m x 4.20m structural span. The system is a skeletal structure composed of prefabricated three-storey, two-storey and one-storey 38cm x 38cm columns and ceiling slabs. All prefabricated ceilings are ribbed slabs with a finalized ceiling. The total thickness of the ceiling slab is 22cm. The cassettes are filled with expanded polystyrene (EPS) boards 15cm thick as a lost formwork. Cantilever slabs are 150cm wide. The skeletal structure is stiffened with prefabricated RC shear walls d=15cm except at the levels of the technical floor and the roof structure where they are cast in-situ. Façade walls, parts of loggia fences, jardinières, bathrooms, as well as toilet cabins are also prefabricated [29].

Figure 4. Analysed freestanding high-rise residential buildings

Two types of external building walls towards the heated space are analysed (Figure 5):

- type A – facade panels, and
- type B – facade walls within the loggia.

Facade panels are prefabricated concrete panels with expanded polystyrene as thermal insulation. All the main parts of the façade are made of these multi-layered concrete panels except the walls of the non-residential part of the ground floor and attic floor, which are cast in-situ. The façade walls in the loggia are made of gas concrete blocks. For both types of walls heat transfer coefficients were calculated and are presented in Figure 5.

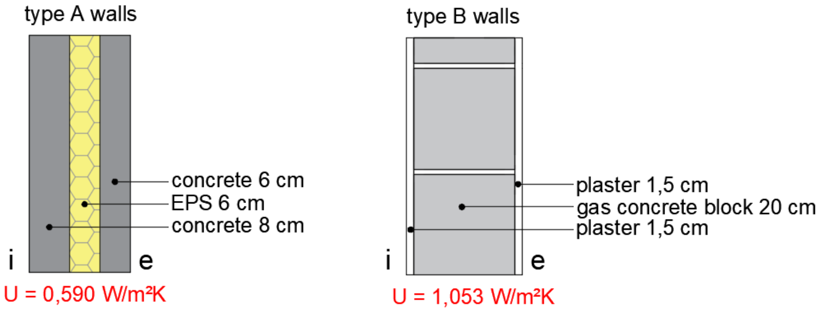

Figure 5. Analysed external building walls: type A – facade panel, type B –wall within the loggia

As it can be seen from the Figure 5, analysed façade walls do not meet the current thermal protection regulations (U_{2012}<0.4 W/m^{2}K). Therefore, in order to improve energy efficiency of the building it is necessary to reduce the heat transfer coefficients which can be achieved by installing an additional thermal insulation layer on external side of the walls.
Apart from improving the energy efficiency of the building, the installation of external thermal insulation on the building facade has other advantages:

- it protects the existing façade elements and significantly slows down the process of deterioration;
- it prevents water vapour condensation and allows the accumulation of heat in the walls, contributing to the thermal comfort as well;
- it allows continuous thermal insulation of the facade, thus avoiding the occurrence of thermal bridges, etc.

However, installation of additional thermal insulation layer on external side of the walls requires the use of scaffolding which significantly increases costs in regard to installation of insulation on internal side of the walls.

In order to meet required fire safety criteria (Table 1), only non-combustible thermal insulation materials were considered in selection of thermal insulation materials. Physical properties of selected thermal insulation materials are presented in Table 4.

Table 4
Physical properties of the thermal insulation materials used in the case study

THERMAL INSULATION MATERIAL	Stone wool	Glass wool	Cellular concrete
Density (kg/m^3)	100	21	115
Thermal conductivity (W/mK)	0,035	0,034	0,045
Specific heat (kJ/kgK)	1	1	1,3
Water vapour diffusion resistance factor	1	1	3
Fire classification	A1	A1	A1

In addition to its low thermal conductivity, *mineral wool* has high resistance to moisture damage, good acoustic properties, it is non-flammable and it will not melt until temperatures reach beyond 1000 °C [30]. *Cellular concrete* is an ecological, mineral insulation material based on the raw materials sand, lime, cement, and water. It is non-flammable, dimensionally stable and prevents mold problems. Material is breathable so moisture is temporarily stored, then re-released into the ambient air. It is also non-combustible and free of toxic emissions [31].

For both types of façade walls, three renewal solutions were proposed (Figures 6-8):

- **Variant I**: ETICS with stone wool (A-I and B-I walls)
- **Variant II**: panel system with glass wool (A-II and B-II walls)
- **Variant III**: ETICS with cellular concrete (A-III and B-III walls)

In order to meet the new thermal protection requirements, the minimum required thickness of thermal insulation layer was determined for all three variants and for both types of façade walls. Improved heat transfer coefficient values are shown in Figures 6-8.

In variant I (Figure 6), ETICS is composed of a hard stone wool panels, fixed to the substrate through bonding products and mechanical fasteners. A thin, reinforced layer is applied over the insulation, and an additional final protective layer. The calculations showed that for type A walls 3 cm of glass wool and 6 cm for type B walls is sufficient to achieve a satisfactory heat transfer coefficient.

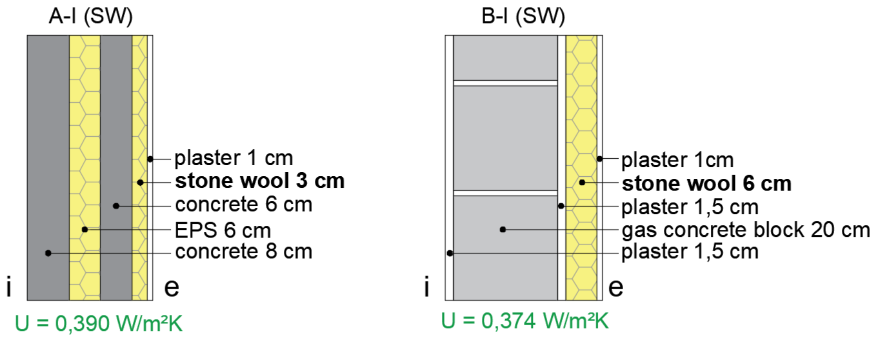

Figure 6. Proposed façade system with stone wool – variant I

In variant II (Figure 7), façade system is composed of a metal substructure, semi-rigid glass mineral wool insulation panels and a waterproof, non-combustible cement board reinforced with fiberglass mesh, made of pure mineral lightweight concrete. First, a metal sub-structure is placed on the existing wall through mechanical fasteners, after which the glass wool is placed between the substructure supports and fixed to the substrate in the same manner as stone wool in the variant I. In the last step the cement boards are fasten to substructure and a final decorative layer is applied. The calculation showed that for A walls 5 cm of glass wool is sufficient to achieve a satisfactory heat transfer coefficient. However, as the thickness of this layer is conditioned by the substructure dimensions, consequently a 6 cm was chosen for both types of walls.

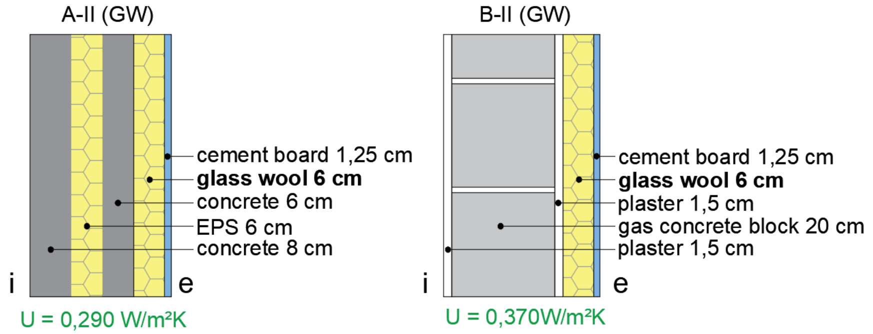

Figure 7. Proposed façade system with glass wool – variant II

In variant III (Figure 8), ETICS is composed of a cellular concrete panels and the installation is similar to variant I installation. The calculations showed that for type A walls 5 cm of thermal insulation and 7.5 cm for type B walls is sufficient to achieve a satisfactory heat transfer coefficient.

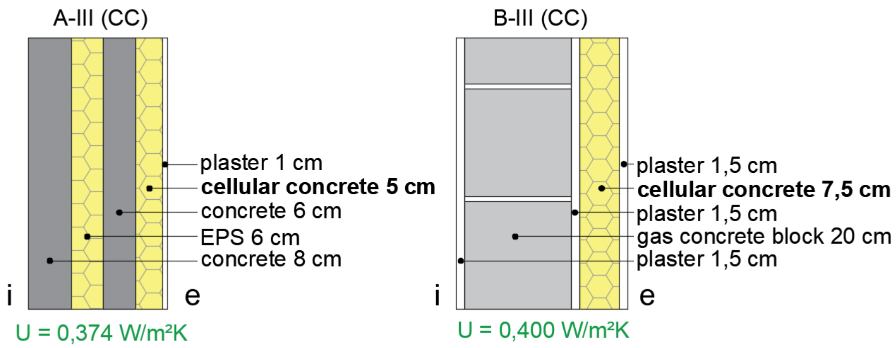

Figure 8. Proposed façade system with cellular concrete – variant III

Within the last phase the proposed solutions were analysed in the context of:

1. application on existing walls,
2. vulnerability to mechanical damages,
3. resistance to wind action, and
4. costs of selected thermal insulation materials.

1. Visual inspection of the façade walls revealed damages, such as spalling, flacking and cracking. Thus, assessment of the substrate onto which the ETICS will be applied should be carried out, as well as substrate preparation. There are several methods for the testing of substrate suitability for the ETICS application, as well as measures to be carried out on the existing walls before its application [32]. Installation of thermal insulation in variant II does not require special substrate preparation, which gives an advantage to this system over the other two systems.
2. The integrity of proposed systems is essential to achieve the desired building efficiency and to obtain the expected level of performance [33]. Even minor mechanical damage (caused by acts of vandalism, hail or other) to the finishing ETICS coat makes it easier for

the water to infiltrate the enclosure and lead to moisture accumulation within the façade. It speeds up the process of its degradation and any possible accumulation of moisture in the thermal insulation coat reduces the effectiveness of the thermal insulation [34]. In this context, façade system II has advantage over the other two systems, thanks to cement boards that protect it against external influences.
3. Wind action represents the main load that the façade must withstand. ETICS shows some problems regarding the operational reliability, being vulnerable to wind action. Pulling off ETICS from the building façade is most of the time caused by the failure of the adhesive bond between the mortar and the wall [35]. In this context, variant II represents the optimal solution with remark that dimensions and cross-sections of the substructure must be matched to the effects of wind pressure and wind suction [36]. An additional analysis of the wind effect on the proposed systems should be carried out in order to ensure their stability and reliability.
4. In addition to the cost of thermal insulation material, the overall costs also include the costs of other façade system components (bonding products, mesh, anchor, bolts, plaster, etc.), as well as the cost of installation, which depend on the building type and complexity, its location, accessibility, condition of substrate and other. Assuming that the installation costs and cost of other façade system components for all three proposed solutions would be similar, only the costs of insulating material were analysed, as well as cost of additional layer (cement board) which appears in the variant II (Table 5).

Table 5
The estimated costs for analysed thermal insulation materials and cement board used in variant II

Façade system		**V-I**	**V-II**		**V-III**
Material		Stone wool	Glass wool	Cement board	Cellular concrete
Type A walls	Layer thickness	3 cm	6cm	1,25 cm	5 cm
	Cost	4 €/m^2	5,5 €/m^2	20 €/m^2	8 €/m^2
Type B walls	Layer thickness	6 cm	6 cm	1,25 cm	7,5
	Cost	6,5 €/m^2	5,5 €/m^2	20 €/m^2	16,5 €/m^2
Rank of proposed solution		**1**	3		2

Façade system V-II is the most expensive of all proposed solutions as requires the use of non-combustible panels that would support glass wool that is being produced in rolls or in the form of semi-rigid panels and, therefore, it cannot be applied within ETICS. This variant is almost 5 times more expensive than variant V-I and twice as expensive as V-III variant, while variant V-III is over 2 times more expensive than V-I variant. Price analysis imposes stone wool (V–I façade system) as the optimal choice for energy rehabilitation of façade walls.

5. CONCLUSION

Potential energy savings in the building sector are greater than in any other sector. Consequently, improving the energy performance of buildings is crucial for reaching the EU's 2030 climate and energy targets.

One of the most common measures for increasing the energy efficiency of buildings is installation of an additional thermal insulation layer on facades. Insulating materials must guarantee acceptable performance throughout the whole life cycle of the building, but thermal performance is not the only parameter that should be addressed when selecting an insulator [37]. If the problem is viewed comprehensively, choosing the best solution requires analysis from several different aspects and the implementation of an adequate optimization method.

In recent years, the number of façade fires in high-rise residential buildings increased, mostly because of the use of combustible materials and nonconformity with prescribed standards and legal regulations. Therefore, the selection of materials for the energy rehabilitation of façade must not impair the fire safety of the building as design and materialization of façade elements belong to a group of key factors for the fire development and fire spread.

More strict requirements for energy efficient buildings, renewal of existing building stock, application of new materials and especially the latest facade fire events in the world [38] initiate the need for a more detailed fire safety analysis of facades.

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CONCLUDING REMARKS

The building industry is constantly innovating in pursuit of environmental, economic and construction efficiency. It is the role of the fire safety engineer to understand, anticipate and react to these innovations to ensure that disaster is avoided.

During the K-FORCE project, we successfully designed (or innovated) and implemented 6 master programs in Disaster Risk Management and Fire Safety field, at higher education institutions in Albania, Bosnia and Herzegovina and Serbia. To ensure the sustainability of these programs – to educate the future educators and scientists, we also established multidisciplinary PhD studies at Faculty of Technical Sciences, University of Novi Sad, Serbia. The new designed LLL courses are aimed to update the professionals in the field. This K-FORCE joint venture also resulted with high level of cooperation between project partners from EU and the Balkans, in order to provide the best possible education model and learning material.

Fire Safety Engineers design systems and structures that lie dormant within our built environment, waiting for an extreme event. The knowledge, technology and engineering that contribute to fire safety are fundamental to escaping occupants, the fire and rescue service, and the protection of property. To meet this challenge, fire safety engineers need to be equipped with fundamental knowledge and experience with fire phenomena, and an understanding of how people, structures, and fire safety systems respond to fire. Fire engineers must be able to combine these diverse fields of knowledge to create infrastructure that is inherently safe, while meeting the needs of clients, architects and fire authorities.

Fire safety engineers have always been in great demand by industry, insurance companies, rescue services, educational institutions, consulting firms and government bodies around the world. We hope our students will continue to advance and improve during their professional careers and to build fire resilient structures and society.

The editors

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REVIEW EXCERPT

The textbook *Fire safety of buildings, A Western Balkan approach and practice*, covers all the important fire safety aspects. The measures that need to be taken in order to achieve the required level of fire safety of buildings are defined, and also the basics for design of structures exposed to fire, as an important aspect of fire engineering, are given.

Considering that fire, in accordance with the recommendations given in Eurocodes for structural design, has become an action that has to be taken as incidental load on structures, this textbook gives the properties of building materials at elevated temperatures, as well as the fire design procedures for concrete, steel and timber structures exposed to fire.

The problem of possible damages caused by fire is especially treated, as well as methods of assessment and rehabilitation that are adequate for these damages. Finally, the problem of fire safety of energy efficient buildings is especially treated, with a special emphasis on energy renovation of facades.

Considering the content, this textbook fully covers the syllabus of the subject Fire Safety of Buildings and is intended for fire safety engineers, structural engineers and architects as well. The student-friendly presented content with illustrations and photographs, as well as included summary of legislation relevant to fire safety, makes this textbook an important literature in the field of Fire engineering.

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