Structural Fire Behaviour

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Abstract. Fire safety and fire protection objectives require that buildings and parts of buildings do not collapse during a fire. This requires that the load-carrying capacity is maintained to a minimum acceptable level during a fire. This chapter briefly describes the historical background and state of the art of fire resistance and its determination for timber members through testing or calculations. The thermal and mechanical principles that underpin structural behaviour of wood at elevated temperatures are explained in the context of explicit calculation methods that enable explicit evaluation of the structural capacity beyond fire resistance, which is a formalised and codified assessment of structural elements against a standard fire. The importance of connections to the overall structure in fire is explained along with suitable design considerations. Ultimately, knowledge gaps with respect to novel and more complex engineered timber products for taller timber buildings are highlighted alongside potential limitations of established design parameters.

Keywords: fire resistance · fire safety · structures · load-carrying capacity · timber · connections

1 Fire Resistance

In the past, catastrophic fires were often driven by the spread of fire, and structural collapse was considered a consequence of out-of-control fires. Thus, structural fire safety was regulated implicitly through the imposition of non-combustible materials, for example in the rebuilding of Rome and London after their great fires in 64 AD and 1666, respectively.

The increased awareness of the importance of individual buildings, paired with an increased societal valuation of life led to the recognition that structures and their individual structural elements play important roles to achieve life safety and to prevent catastrophic fire spread. Preventing structural collapse and constraining a fire to its compartment of origin can reduce deaths among occupants and emergency personnel and limit material losses. This led to the development of fire resistance [1, 2] to provide a

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means to compare different building assemblies measured against criteria to limit the conduction of heat, prevent the passage of smoke or flame, and maintain load-carrying capacity throughout a fire, which this chapter focuses on. The former two may also be fulfilled by non-load-carrying building elements.

The initial concept of fire resistance was to assess performance against the three criteria above (load-carrying capacity, integrity, and insulation) to the equivalent duration of a full burn-out fire [3] this means that collapse should never occur in a fire with a finite specified fuel load. Modern codified guidelines and regulations specify fire resistance not only with respect to the fuel load but also accommodate the risk profile of a building. For example, even if a fire has equal physical consequences for a structural element, that element would require a higher fire resistance rating in a high-rise hotel, compared to a single-family house. This takes the expected direct and often indirect losses into account, as well as the increased difficulty to fire-fighting operations that arise in taller buildings.

The assessment of fire resistance is done against a so-called 'standard fire'. This can be done experimentally, in a furnace, or using current modelling and design procedures. Multiple variants of standard fires exist, although the most common one is described as the standard temperature-time curve, ISO 834, that represents the evolution of the temperatures in a compartment engulfed in a fully developed fire. It prescribes a defined rise in temperature and pressure conditions, although the means of measuring this rise vary between jurisdictions. For example, in North America, thermocouples measure the apparent gas phase temperature, while in Europe, plate thermometers are prescribed to measure the adiabatic surface temperature. Thus, while the concept is standardised, the outcomes of fire resistance tests in one jurisdiction do not guarantee acceptance in others.

Standard fires are not supposed to replicate real fires but rather provide a standardised exposure against which fire resistance can be assessed. It is important to remember that fire resistance duration is decoupled from time in a real fire. Fire resistance and heat exposure in standard fires are only comparable to the conditions in a fully developed, under-ventilated fire. Alternative time-temperature curves may be used to better reflect specific compartments [4], for example, parametric fire curves account for ventilation and heat losses through compartment geometries to account for different compartment fire heating rates and a fire decay phase as fuel burns out.

2 Effect of Temperature on Mechanical Properties

All materials experience a change in their mechanical properties in response to a change in temperature. Elevated temperatures cause a deterioration of mechanical properties through a variety of mechanisms. In wood, both strength and stiffness are affected by physical changes, often associated with the movement and storage of water, and through the thermo-chemical decomposition of the polymers that make up the majority of its biomass. Depending on the specific species, wood is generally composed primarily of cellulose (40–50%), hemicelluloses (20–30%), lignin (20–30%), and other minerals and extracts [5]. Like other construction materials, wood undergoes various thermal decomposition processes at elevated temperatures, resulting in both physical and chemical changes. At temperatures up to 100 °C, the water within wood is displaced, diffuses, and/or evaporates, causing shrinkage. At temperatures exceeding 200 °C, the hemicelluloses and lignin begin to decompose and release combustible gases; this thermal decomposition process is known as pyrolysis. Above 280–300 °C, the cellulose compound starts to decompose and residual carbonaceous char is formed during pyrolysis. In the presence of sufficient oxygen, charred wood undergoes further degradation due to oxidation reactions, typically occurring in the temperature range of 400–500 °C [6].

From a structural perspective, the cross-sections of fire-exposed timber elements show reduced mechanical properties in the areas under elevated temperatures and no load-carrying capacity once it has charred. The structural performance of timber elements in fire is most commonly and most directly linked to the concept of charring. Charring is the thermal decomposition process that forms residual high-carbon char upon pyrolysis. This process generally occurs at temperatures above 280–300 °C and significantly affects the mechanical properties of timber. Beyond 300 °C, the conversion of wood to char is typically assumed to be complete, and the resulting charred wood is considered to have negligible strength and stiffness [7]. In particular, the 300 °C isotherm is often used to define the location of the char front. By tracking its progression, it is possible to estimate the char depth and charring rate, which are crucial parameters for assessing the load-carrying capacity of timber elements exposed to fire.

After estimating the effect of charring, it is essential to understand the impact of heat penetration in uncharred heated wood to evaluate the actual load-carrying capacity of timber structural members exposed to fire. As mentioned, timber decomposition processes at temperatures below the charring threshold also lead to significant reductions in mechanical properties. Figure 1 illustrates typically assumed reduction factors for strength and modulus of elasticity parallel to the grain for typical softwood exposed to a

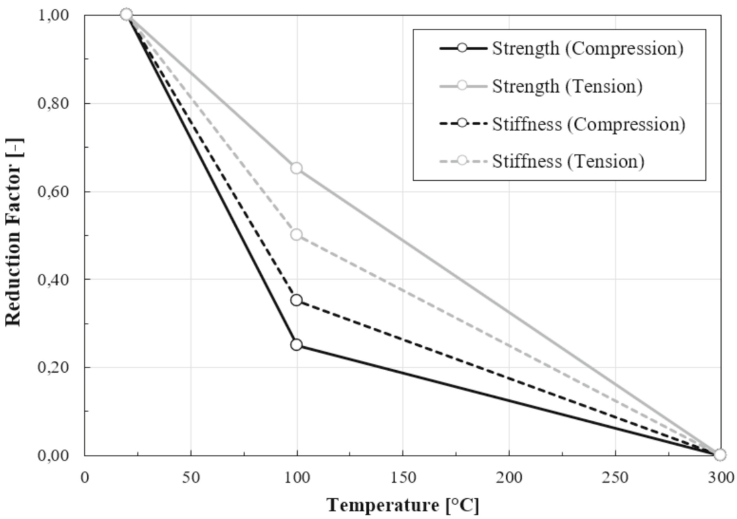

Fig. 1. Reduction factor for strength and stiffness (modulus of elasticity) parallel to grain of softwood exposed to the standard fire curve (after [8]).

standard fire, according to Eurocode 5 Part 1-2 [8]. For example, at 100 °C, the compression strength of timber is assumed to be reduced by 75% compared to its value at normal ambient temperature, while its tensile strength is reduced by 35%. However, given the low thermal conductivity of timber, the temperature gradient within fire-exposed cross-sections is very steep, at least during the fully developed phase of a fire. This means that underneath the external charred layer, the temperatures drop in short distance, often keeping the core of the cross-section at relatively low temperatures. In the cooling phase of a real fire, however, this gradient tends to flatten and elevated temperatures tend to homogenise inside the cross-section.

3 Current Assessments of Fire Resistance and Structural Fire Capacity

Fire resistance requirements can be met either by the use of standard constructive solutions, through testing, through established design models, or more complex simulations. These design models consider the reduced mechanical properties of the char layer and heated layer of uncharred timber. Under standard fire exposure, the charring rate of timber cross-sections converges to a quasi-constant value once a stable char layer is formed after approximately 20 minutes and the temperature profile beyond the char stabilizes [9]. This nearly constant char rate and steady-state temperature profile underneath the char front is the basis of the effective (or reduced) cross-section method, which subtracts a char layer and a so-called zero-strength layer from the initial cross-section. Variations of this method are implemented in design standards globally.

For a calculation of the char layer thickness for wood under standard fire exposure, a one-dimensional charring rate of 0.65 mm/min is generally assumed for softwoods; this is increased to account for corner rounding in multidimensional exposure conditions. For hardwoods or high-density softwoods, lower rates may be expected. As the strength of the char layer is negligible, it should be fully subtracted from the cross-section for structural calculations. This, however, does not hold for the heat-affected zone. Instead of calculating the structural capacity using the steep temperature gradient and the corresponding temperature-adjusted mechanical properties, which are lowest near the char front and highest deeper into the member, the use of the zero-strength layer [10] simplifies calculations and assumes a thinner uniform layer with zero strength underneath the charred layer, while the remaining cross-section is assumed to maintain the full initial capacity (Fig. 2). Several design guidance documents assume a constant zero-strength layer of 7 mm, which is based on the work of Schaffer on bending members [10]. Although rarely implemented in practice, numerical methods can be used to determine structural fire resistance, where simpler methods are not sufficient. Such numerical methods are usually based on uncoupled thermal-mechanical analyses, in which the temperature fields are calculated at successive fire exposure times and the temperature-dependent mechanical properties are adjusted accordingly. This approach allows for determining the stress fields and the mechanical behaviour during the fire exposure and assumes that the stress fields do not influence the temperature distribution, which is not always valid, particularly in the case of timber connections.

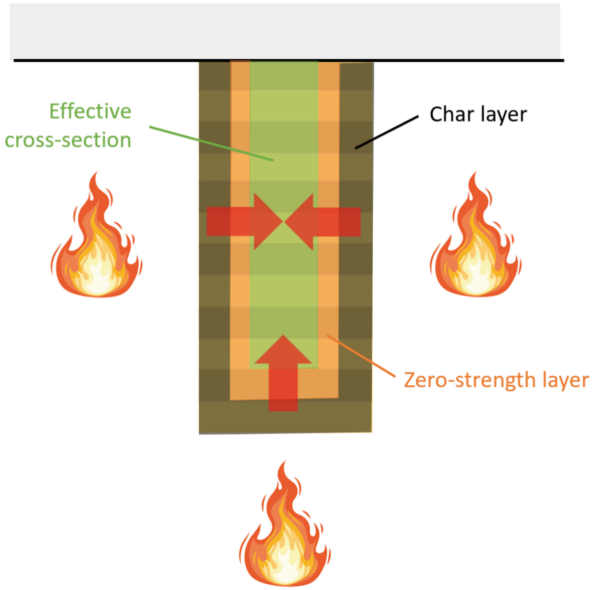

Fig. 2. Schematic showing char and zero-strength layers (ZSL) defining the effective cross-section in a mass timber beam subjected to fire from three sides. Arrows denote progression of char and ZSL fronts.

The temperature calculations can be done using, for example, non-linear finite difference or finite element heat transfer calculation methods. Most heat transfer calculations rely on effective thermal properties that implicitly take into account multiple complex phenomena, such as pyrolysis, fissures and associated mass transfer. These effective thermal properties are intended to give good approximations of the temperature fields of timber exposed to the standard fire exposure.

For structural calculations, numerous proposed relationships link temperature with the strength and stiffness of wood under compression, tension, and shear loading [11–13]. These relationships vary significantly among authors, which can be attributed to the differences in outcomes that arise between tests in steady-state or transient temperature conditions [7, 14]. Relationships provided by Eurocode 5 [8], shown in Fig. 1, were determined from transient conditions in standard fire exposure; while steady state conditions result in less onerous reductions, most fire situations will induce transient conditions. The use of advanced thermal or thermal-mechanical simulations does not guarantee conservative designs, unless the scope of application and limitations of such models are strictly considered.

4 Connections

Connections often govern the design of timber structures at normal ambient temperature, namely because of the limited load-carrying capacity and stiffness of common connections compared with the timber members being joined. Connections with metallic dowel-type fasteners (nails, staples, dowels, bolts, screws) and plates are efficient and widely used in timber structures. At normal ambient temperature, the serviceability and load-carrying capacity of timber connections depends on the type, geometry, and layout of the fasteners, on the angle between the loading direction and the axis of the fastener, and on the embedment, withdrawal, shear, and splitting properties of the connected timber members.

In fire, the performance of timber connections is even more critical for the behaviour of timber structures. This is namely due to charring of fire-exposed timber members, increased heat transfer into the timber members through the metallic fasteners and plates, high localized stresses close to the fasteners, heat transfer through gaps and slots, and the already mentioned temperature dependency of mechanical properties.

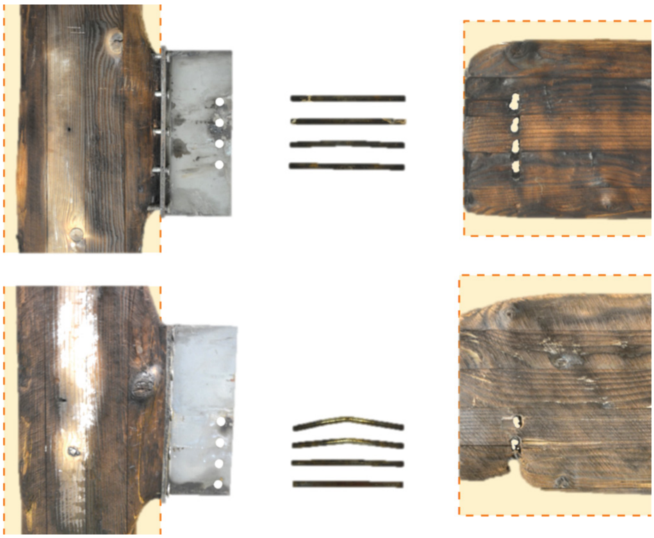

Fig. 3 Beam-to-column connections after fire resistance testing [15].

Given the complex mechanical and thermal interactions that take place in timber connections during fire exposure, the study of their structural fire behaviour, particularly the prediction of their fire resistance, has been heavily based on experimental research [15–22](Fig. 3). Even though many experiments have been performed on connection configurations that are not typical in taller timber buildings, many findings still apply. The most important parameters that influence the fire resistance of timber connections are the load level (ratio between the load applied in the fire situation and the load-carrying capacity at normal ambient temperature), the thickness of the timber side members, the width of gaps and slots in the connection area, the type of fastener and how much the steel parts protrude from the timber members, the end and edge distances of the fasteners, and the fastener spacing. Metallic parts and wide gaps or slots increase the heat transfer into the timber members, leading to increased charring in the connection zone. The most common design methods are based on protecting the connection through increased member cross-section and fastener end and edge distances and spacing or by attaching panels to cover the connection zone (the latter being aesthetically not always very appealing). Gaps and slots can also be filled or covered to limit heat transfer. This means that most fire resistance requirements can the fulfilled through an adequate choice of connection geometry based on the imposed load level. The upcoming version of Eurocode 5 provides tabulated data for this purpose. Empirical or semi-empirical fire resistance models (function of the type of connection, time of fire exposure or load level, and side-member thickness) are also available, even for fire resistances up to 120 min. Advanced thermal and thermal-mechanical modelling of connections require additional considerations, namely due to the abovementioned complex heat transfer phenomena and because of the influence that the large deformations have on the temperature distribution, as the surfaces between members become progressively more exposed to fire.

5 Challenges to Existing Methods and Knowledge

5.1 Fire Decay and Cooling Phases

Most of the discussed assessment methods aimed at quantifying the load-carrying capacity of timber structural elements exposed to fire traditionally focus on the fully developed phase of post-flashover compartment fires, usually known as the "heating phase." Nevertheless, after this phase, a compartment fire within a building enclosure is followed by decay and cooling phases [23]. Current standardised fire testing and structural fire assessment methodologies do not require investigating the effects of the fire decay and cooling phases because they are traditionally considered less critical due to their lower temperatures and because the fire resistance prescribed by building authorities is assumed to implicitly account for these phases.

During the fire decay and cooling phases heat continues to diffuse within structural materials through conduction. In particular, even if the temperature at the exposed surface is decreasing, the heat wave continues to diffuse and penetrate the structural elements, leading to an internal temperature rise and, consequently, a reduction in the load-carrying capacity of structural elements [24]. Since timber undergoes an irreversible loss of mechanical properties at relatively low temperatures compared to traditional construction materials like steel and concrete (refer to Fig. 1), the penetration of the thermal wave during the fire decay and cooling phases poses a serious challenge for timber structures. The relevance of considering this phenomenon has been recently highlighted by several researchers [25–27], and experimental evidence has shown timber structural elements failing during the cooling phase [28–30]. Research has underlined that in-depth penetration of the heat wave is highly affected by the thermal boundary conditions during the fire decay and cooling phases, as well as the duration of the various fire phases [31]. Typically, the heating phase dominates the development of the char depth, but the cooling phase has a non-negligible contribution. In addition, the uncharred heated timber increases throughout the entire fire scenario, particularly during cooling and possibly even after the end of the cooling phase. Due to the four-sided fire exposure, timber columns are particularly susceptible to this phenomenon [29].

Current structural fire engineering approaches do not address this issue explicitly, and recent research outcomes have highlighted the necessity of incorporating the fire decay and cooling phases in modern performance-based methodologies for fire-safe timber structures. One way to address this issue, at least partially, is to fully encapsulate the structure, however, this can stifle the full potential or cause overly conservative designs in the absence of fully engineered solutions.

5.2 Different Engineered Timber Products

5.2.1 Stress Distribution

Existing structural fire design guides mainly utilise the concept of a zero-strength layer to perform structural calculations for the determination of the fire resistance. This concept originated from work by Schaffer et al. [10] for simply-supported glulam beams in bending. Recent studies indicated that the generally implemented zero-strength layer of 7 mm does not reliably result in conservative predictions of the fire resistance [32]. More significant overestimations of the capacity are made for:

- Timber members that are loaded in compression and prone to buckling
- Non-homogeneous timber members

This is because the mode of loading determines the strain and stress distributions and the stresses near the char front. The capacity of a column that is prone to buckling is more dependent on the strength and stiffness of the wood material close to the char front than beams that are loaded in bending. The upcoming version of Eurocode 5 prescribes different zero-strength layer values for different types of loading.

The simplification implemented by the zero-strength layer can lead to non-conservative errors for engineered timber materials with non-homogeneous cross-sections, such as CLT. This can be the case when the zero-strength layer partially falls within non-load-carrying cross-layers. The zero-strength layer was originally proposed due to its ease of use; if it requires a wide range of values to account for varying scenarios that arise in more complex timber buildings it may be preferable to calculate the load-carrying capacity directly from the temperature gradient and the relationships of temperature and mechanical properties that are shown in Fig. 1.

5.2.2 Adhesive Performance and Composite Action

Recent increased interest and implementation of timber structural elements for tall timber structures have mainly been driven by engineered timber products, of which the most popular types, cross-laminated timber (CLT) and glue-laminated timber, utilize adhesives to form structural elements out of multiple boards.

At normal ambient temperatures, adhesives are expected to provide the same or more shear strength than the wood itself. This ensures composite action so that the assembly of individual timber boards act as one composite structural element to greatly increase the second moment of area and thus the flexural stiffness.

Different performance of adhesives at elevated temperatures have been an important consideration for the fire dynamics in a compartment, especially for CLT, as highlighted in Chap. 21. Even before fall-off of charred wood, elevated temperatures and moisture levels can weaken mechanical properties of adhesives [33–35]. When this weakening reduces composite action, significant reductions in overall structural capacity are possible [28, 36–38].

Different adhesive formulations exhibit different mechanical performance at elevated temperatures and their selection is dependent on the balance of mechanical performance, costs and environmental considerations. In North America, a minimum level of performance of adhesive in fire situations is assessed in PRG 320 [39]. While this test applies loads, the performance requirements for an adhesive to pass are solely based on the occurrence of char fall-off and not on structural performance. Thus, while a product might be suitable from a fire dynamics perspective, the adhesive may still cause a product to have an inferior flexural capacity during a fire.

5.2.3 Smouldering

Smouldering, also sometimes referred to as glowing, describes the oxidation of solid-phase carbon. It is an exothermic reaction and can under some conditions self-sustain. It occurs where airflow can interact with sufficiently hot char and heat losses are reduced, for example near insulation or in corners and crevices. If smouldering is self-sustained it will eventually lead to structural collapse, unless extinguished by fire service intervention. Smouldering progression is markedly slower than surface flame spread and thus fire service intervention after cessation of flaming is possible if smouldering is detected; this is usually achieved from infrared camera inspections and the potential removal of insulation and fire protection. Consequently, most codified guidance documents do not currently consider smouldering explicitly for structural design requirements. An exception is Japan, where for some jurisdictions and building heights, fire resistance tests must be continued after the prescribed fire resistance time has been achieved. Tested building elements must achieve self-extinguishment of smouldering to pass the fire resistance test.

6 Concluding Remarks

The structural fire safety of timber structures has long been investigated and successfully implemented in buildings with a timber frame structural system, mainly via standard fire resistance testing and associated calculation methods. These designs are underpinned by knowledge of the relationship between mechanical strength and temperature and expected temperature profiles in standard fires. However, knowledge gaps exist for fires beyond standard fires. Recent research outcomes emphasise that both the heating and cooling phases play a crucial role in assessing the heat penetration and the consequent reduction of mechanical properties in timber structural elements exposed to fire. Indeed, during the fire decay and cooling phases, the load-carrying capacity of timber elements can significantly reduce due to heat wave penetration (further charring and heated timber), and ignoring this phenomenon can lead to substantial underestimation, resulting in unsafe assessments. In addition, the constant development of novel timber products requires consideration regarding their link to existing design solutions.

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