Design of taller timber buildings subjected to accidental loads: a state-of-the-art review

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Holistic design of taller timber buildings - HELEN

Working Group (WG) 3

Accidental Load Situations

State-of-the-art (STAR) report

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Forward

This report is a publication of the European Network **COST Action CA20139** “Holistic design of taller timber buildings – **HELEN**”, established with the aim to “*work towards optimized holistic approaches to improve the performance of taller timber buildings and to widen their competitiveness and use across the EU and rest of the world*” (https://cahelen.eu/).

The activities conducted in the first year of the Action by the **Working Group (WG) 3** - Accidental load situations - are summarized in this document in the form of a state-of-the-art report (**STAR**) regarding design, analysis and construction methods of taller timber buildings subjected to load situations due to **earthquake**, **fire** and **blast**.

The report is the result of a deep review of scientific literature, international projects, national regulations, design guidelines, as well as case studies. Particular attention has been paid to the potential interactions with other fields of design and to the efforts made in the recent years to overcome the limitations for the progress in the construction market of timber buildings under seismic, fire and blast loads.

The information collected in this STAR document represent the starting point of discussion to identify solutions, research targets, methods and resources for the future of taller timber buildings under seismic, fire and blast loads following a holistic design approach.

Three different sub-groups (SGs) have been defined for WG3 STAR activities, namely **SG1 - Seismic Loads**, **SG2 - Fire** and **SG3 - Blast**. For each SG, different subtopics have been analysed and discussed.

This report is structured into three parts. **Part 1** includes the review conducted by SG1 on the seismic design and seismic analysis of taller timber buildings and consist of seven documents. The review regarding the fire design situations is summarized in **Part 2** through four documents. **Part 3**, composed of four documents, addresses the review of current knowledge on blast design of timber buildings.

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Part 1

Seismic Loads

Sub Group (SG) 1

WG3.SG1.01

Lateral Load Resisting Systems

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1. Introduction

The structural behaviour of Tall Timber Buildings (TTBs) subjected to earthquake actions is governed to a large extent by the Lateral Load Resisting Systems (LLRSs), whose performances are mainly dependent on their geometry, the Engineering Wood Products (EWPs) adopted for different structural elements, and the type of connections used.

An overview of most adopted systems is presented, and associated examples of constructed buildings are introduced.

2. Shear wall system

Shear wall systems used as LLRS for TTBs are mainly composed of Cross-Laminated Timber (CLT) panels. The benefits of such systems are reasonably high in- and out-of-plane stiffness and strength, rapid construction process, and established design provisions (A. Ceccotti & M. Follesa, 2006). Alternatives to CLT are Glued Laminated Timber (Glulam), Laminated-Veneer Lumber (LVL), Laminated-Strand Lumber (LSL), and special mass plywood panels.

In general, there are two types of shear wall constructions, see Figure 1:

- Platform system, in which the floors at each storey are the base of the shear wall for the next level. Hence, each floor is the interval between the shear walls of adjacent storeys.
- Balloon system, in which the shear wall is erected continuously over multiple floor intervals.

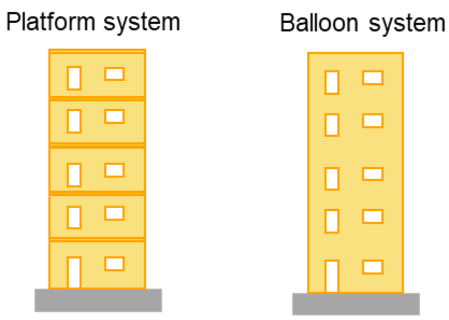

Figure 1: Shearwall construction types: Platform (left) and Balloon (right) system.

Platform system is known as a more dissipative system since more connections are involved, leading to higher redundancy. Capacity-based Design (CD) is incorporated into design of such system in high seismic regions by providing dissipation through relatively ductile connections and over-protecting non-dissipative elements to avoid brittle failure (Casagrande et al., 2019).

Many examples of low-to mid-rise buildings composed of CLT shear walls are built worldwide, such as:

- Bridport house in London, UK (2010); up to eight storeys made of platform-type CLT shear wall.

- Origine in Quebec City, Canada (2017); thirteen storeys made of balloon-type CLT shear wall.

3. Timber frame system

LLRS in timber frame systems may be provided by braces or moment resisting connections, as can be seen in Figure 2a. The ductile mechanism in the connections at the end of the diagonal braces dissipate energy while other members and connections remain elastic (Chen & Popovski, 2020). The connections used in braces may be considered relatively ductile if they are properly designed, such as bolted connections without risk of wood splitting.

Moment resisting connections were rarely used as LLRS due to the difficulty in details. Ductile connections are needed to provide enough energy dissipation while minimizing shrinkage. Some examples of such connections are bolts with tight-fit pins, glued connections, etc.

Examples of timber frames used as LLRS in mass timber constructions are:

- Mjøstårnet in Brumunddal, Norway, (2019); eighteen storeys composed of braces as LLRS.
- UBC Okanagan Fitness and Wellness Centre in Kelowna, Canada (2013); tapered moment-resisting connections were utilized.

Figure 2: Timber frame LLRSs: brace (left) and moment-resisting (right) frame, (a); Hybrid LLRSs: timber-steel (left) and timber-concrete (right) systems, (b).

4. Hybrid systems

Hybrid systems represent those LLRS, in which timber elements are combined with steel and/or concrete elements (Figure 2b). A large variety of hybrid systems can be adopted as LLRS, however, one of the most adopted solutions consist of a reinforced concrete core that fulfils the structural performance and simultaneously is the stair and lift shaft, as is the case with conventional reinforced concrete high-rise buildings. Such a system presents several advantages such as: (i) it eases the limitation related to fire safety of TTBs as stairs and lift shafts are constructed of non-combustible materials, (ii) its performance under seismic actions is significantly less complex to be designed for the majority of practitioners, (iii) it represents a cost-optimized solution compared to other timber alternatives.

Examples of LLRS hybrid systems for TTB are:

- Haut project in Amsterdam, which is one of the tallest timber hybrid buildings with a 73m height (https://www.arup.com/projects/haut).
- Brock Commons Tallwood House, residence building, Vancouver (Canada).
- MEC head office composed of steel-braced LLRS, Vancouver (Canada).

WG3.SG1.02

High-performance connections

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1. Introduction

The primary role of high-performance connections for Tall Timber Buildings (TTB) is transferring forces and accommodate displacements between structural elements. They assure proper stiffness of timber structures, as well as dynamic behaviour features such as ductility and energy dissipation capacity of buildings in seismic areas. In timber structures subjected to seismic events, the exposure of structural connectors to cyclic loads is one of the dominant issues of stiffness loss and irreversible damages. Therefore, the choice of a proper connection system is generally made by considering not only the strength, but also other mechanical properties such as the stiffness, the ductility and the capacity of dissipating energy, which are all properties influencing the response of TTBs during a seismic event. The properties, geometry and mechanical behaviour of connections for TTBs are primarily related to Lateral Load Resisting System (LLRS), in which they are intended to be used. Considering the main LLRSs adopted for TTBs shown in Figure 1, two main connection categories can be identified: (i) connections for panel elements, and (ii) connections for linear elements. The former serves to connect timber panels to the storeys below and/or to the foundation and to transfer the horizontal and vertical forces due to the seismic actions, see Figure 2 (left). The latter serves to transfer the bending moments and the internal axial and shear forces due to the seismic actions, see Figure 2 (right).

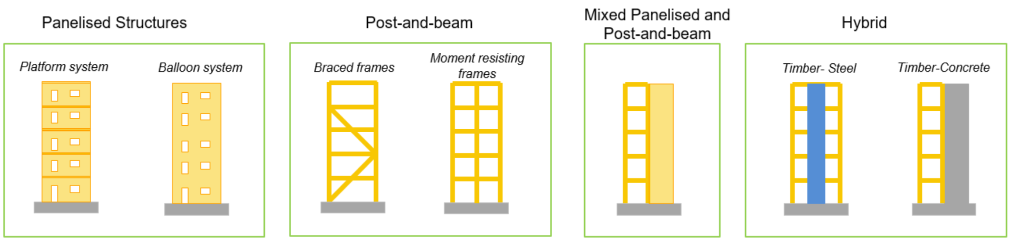

Figure 3: Different LLRS for TTBs.

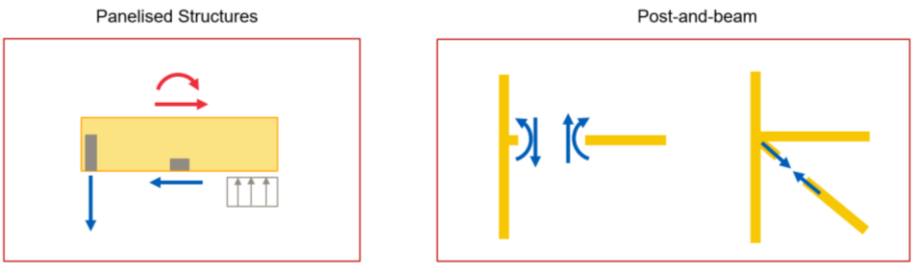

Figure 4: Internal forces in connections of different LLRS for TTB.

Due to their crucial role in seismic performance of timber buildings, connections in timber structures have been the subject of extensive research and development over the last decades and have undergone technological progress that has led to an increase in their mechanical performance and enabled the construction of modern TTBs. In the next sections, an overview of traditional and innovative high-performance connections is presented.

2. Traditional connections

Traditional connections of timber structures in seismic areas were primarily made of carpentry type joints, whereas nowadays they are generally metal dowel-type connections with fasteners such as nails, bolts, dowels, screws or specially formed metal connectors such as toothed-plates, ring and shear plates, metal brackets of various shapes like hold-downs and angle brackets (Porteous & Kermani, 2007)). Stiff and high strength adhesive type connectors joining structural elements such as composite beams with timber-to-timber or timber-to-concrete joints and glued-in steel rods (Borgström, 2016) are used primarily for non-dissipative connections. Traditional metal type connections adopted in low-rise cross-laminated timber (CLT) structures were inherited from light-frame timber (LFT) structures and include metal brackets and dowelled slotted-in steel plates of various geometries used to undertake vertical-tensile and horizontal-shear forces, and self-tapping partially threaded screws used to transfer shear forces (Tomasi and Sartori, 2013, Gavric et al., 2015, Izzi et al., 2018)). These types of connections were proven adequate for low-rise CLT buildings when accepting a certain degree of damage in the joints and residual deformation in the timber assemblies. When structural designers try to adopt traditional connection systems in TTB, they might encounter difficulties that reveal some relevant issues such as insufficient strength and stiffness capacity, and possible brittle failure modes with the lack of energy dissipation. To overcome these limitations, development of innovative high-performance connections is crucial.

3. Innovative connections

To overcome common drawbacks of traditional connections for timber buildings in seismic areas, such as stress concentration and incompatibilities under large deformations (Moroder, 2016), insufficient cyclic resistance, ductility and energy dissipation capacity, different typologies of innovative high-performance connections have been developed in the last years. Several innovative solutions are available, such as tube connections (Schneider et al., 2018), X-Rad (Polastri et al., 2017)), Spider (Maurer et al., 2021), bi-directional behaviour metal plate connections (D'Arenzo et al., 2021), hyper-elastic hold downs (Asgari et al., 2021), pinching-free connectors (Chan et al., 2021), U-shaped flexural plates (Chen et al., 2020), polyurethane thick flexible shear and glued-in steel rod joints (Pečnik et al., 2021, Azinović et al., 2018), which are durable under fatigue tests (Kwiecień et al., 2019). Most of the abovementioned innovative connections represent solutions adopted to improve the structural behaviour of traditional connections, ensuring higher strength, stiffness, dissipation capacity and overcoming issues related to brittle failure mechanisms. On the other hand, some of the presented innovative connections differ substantially from traditional connections and their adoption in some cases results in a different seismic behaviour of structural timber systems.

4. Energy dissipation connections

Various ideas improving energy dissipation properties of connections were also investigated and proposed in the recent years. Slip-friction devices, consisting of a steel plate encased in the timber element and two built-in abrasive-resistant side steel plates held together with bolts and disc springs, were proposed as a hold-down for mass timber shear walls (Loo et al., 2014). High dissipation energy capability can be attained in the system without pinching, displaying the desired flag-shaped hysteretic behaviour. Slip-friction connectors were further advanced into resilient slip friction (RSF) connectors, which provide a damage-free self-centring solution for CLT shear walls (Hashemi et al., 2018). The self-centring capacity is enabled by the zigzag-like connection interface between the cap and slotted plates. High-force-to-volume damping devices (HF2VDDs) made of a steel shaft sliding within a tube (Vishnupriya et al., 2018) were also proposed. The damping and energy dissipation are provided by an extruded lead mounted around the shaft. Dickof et al. (2021) proposed Internal Perforated Steel Plates as end-brace connections in timber frames and as base shear connectors in CLT shear walls to avoid the common dowel yielding and wood crushing failure mechanisms.

WG3.SG1.03

Seismic Protection Technologies

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1. Low damage and self-centring systems

The low-damage systems and self-centring concepts are based on automatic mechanisms for returning the main irreplaceable parts of structural systems to their initial positions after earthquakes. Therefore, several innovative systems and models have been proposed to increase the self-centring capacity of Taller Timber buildings (TTBs). Table 1 summarizes low-damage and self-centring systems.

Table 1: Low-damage and self-centring timber systems literature review

System type	Author	Year
Resilient Slip Friction Joint (RSFJ)	Zarnani & Quenneville	2015
Self-Centring Rocking timber Wall system (SC-RW)	Hashemi et al.	2016
Hybrid steel-timber wall system using the RSFJs	Hashemi et al.	2017
RSFJ as hold-downs in rocking CLT walls	Hashemi et al.	2018
Self-Centring Steel-Timber Hybrid Shear Wall (SC-STHSW)	Cui et al.	2020
Self-Centring Timber Brace (SC-TB)	Yousef-Beik et al.	2020
Elliptically profiled CLT walls	Ricco et al.	2021
Rocking CLT walls with the Uplift Friction Dampers (UFD)	Tatar & Dowden	2022

Zarnani and Quenneville (2015) proposed a damage-free connection which is called Resilient Slip Friction Joint (RSFJ). The RSFJ has two middle and two cap serrated plates which are compressed by a bolt and disc spring system. The RSFJ has been used by Hashemi et al. (2017) in hybrid damage avoidant steel-timber wall systems to prevent residual displacement and minimize damage. Hashemi et al. (2018) utilized RSFJ as hold-downs in rocking CLT walls to reduce the residual displacement and improve the self-centring capacity of the rocking CLT walls. An Uplift Friction Damper (UFD) composed of tension bolts and disc springs combined with an angled abrasive friction interface has been used as a low-damage and energy dissipater connector in rocking shear walls (Tatar and Dowden 2022). In the proposed UFD, post-tensioned (PT) rods provide the self-centring response while two angled steel wedges slide on each other to dissipate energy. Ricco et al. (2021) proposed using elliptically profiled CLT walls to form a rocking story in multi-story CLT buildings. The system is a rocking soft story based on the elliptical rolling rod isolation. Using soft story as seismic isolator buffers the upper stiff part of the building from ground motions during earthquakes.

2. Post-tensioned systems

Post-tensioned timber technology can provide increased strength and stiffness for mass timber seismic load resisting systems while also providing energy dissipation and re-centring capabilities. The state-of-the-art research and implementation of the post-tensioned timber systems was reported by Granello et al. (2020) and some key research is listed in Table 2. The research shows that the controlled rocking mechanisms of elastic post-tensioning provide self-centring action to eliminate residual drifts, while additional dissipative devices, such as replaceable steel fuses, increase the damping and reduce lateral displacements of multi-storey buildings. The research on the stiffness of connections is still limited, health monitoring and maintenance also require further investigations. Furthermore, the interaction between structural and non-structural elements under seismic loading requires further considerations to achieve a holistic design (Brown et al. 2022).

Table 2: Post-tensioned timber systems literature review

System type	Relevant research
Post-tensioned timber shear walls	1. Laminated veneer lumber (LVL) walls (Palermo *et al.* 2005) 2. CLT walls (Pei *et al.* 2019, Brown *et al.* 2022) 3. Single and coupled Post-Tensioned CLT (PT-CLT) walls (Akbas et al. 2017) 4. Post-Tensioned CLT Shear Walls with Energy Dissipators (Chen et al. 2020)
Post-tensioned timber frames	1. Beam-column connections (Iqbal, Pampanin, and Buchanan 2016, Li, He, and Wang 2018) 2. Frames (Newcombe, Pampanin, and Buchanan 2010, Di Cesare et al. 2020) 3. Post-tensioned glulam frame (Ding et al. 2021)

3. Supplemental damping systems

Seismic protection through supplemental damping aims at decreasing the structural demands and consequently decreasing inter-story drifts while potentially adding stiffness in the structural systems by increasing inherent damping dissipation through the addition of supplemental devices called dampers. These devices can increase the equivalent viscous damping and are activated by: (i) displacement (e.g. metallic dampers, hysteretic devices, friction dampers); (ii) velocity (e.g. viscoelastic dampers), or (iii) motion (e.g. tuned mass dampers). The summary of state-of-the-art research is summarized in Table 3. Applications in-between gravitational and lateral force resisting systems of mid- to high-rise timber buildings may be increasingly exploited in the future, which also applies to rocking systems, yet so far most of research has been performed for low- and mid-rise structural systems. Clear definitions for the calculation of overstrength for timber assemblies, as well as detailing of connections to the gravity system is greatly needed in codes.

Table 3: Supplemental damping systems literature review

System type	Relevant research
Metallic dampers and Friction dampers Viscoelastic and fluid viscous dampers Tuned mass dampers	1. Seismic protection technologies for timber structures (Ugalde et al. 2019) 2. Damping in Wood-Frame Shear Wall Structures (Jayamon et al. 2018) 3. Base Isolation and Supplemental Damping Systems for Seismic Protection of Wood Structures (Symans et al. 2002)

4. Passive control systems

With the new challenge for designers to accommodate the high load demand and ductility, innovative dissipating devices have been combined with mass timber structures as passive control systems as listed in Table 4. The passive control system can minimise damages to timber elements when providing enough structural performance by using capacity design principles. Residual deformation of structures with passive control systems is an issue for the reuse of structures after a major earthquake. Unlike those passive control systems for steel structures, many mass timber structures with passive control systems are not in current standards. Design information is missing such as force reduction factors and ductility factors.

Table 4: Passive control systems literature review

System type	Relevant research
Timber frame with buckling restrained braces (BRBs)	1. Timber casing BRBs (Murphy, Blomgren, and Rammer 2019, Takeuchi *et al.* 2022) 2. Frame with BRBs (Dong *et al.* 2020)
Mass timber with perforated steel fuses	1. Shear wall application (Blomgren et al. 2018) 2. Brace application (Daneshvar et al. 2022)

Mass timber with Resilient Slip Friction Joint (RSFJ) connections	1. Hold-downs for shear walls (Hashemi, Zarnani, and Quenneville 2020) 2. Braces for frames (Yousef-beik *et al.* 2021)

WG3.SG1.04

Design strategies and seismic analysis

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1. Introduction and background

The seismic design process of taller timber buildings typically involves two main aspects: "design strategies" and "seismic analyses". The former deals with the design methodology used to achieve adequate performance levels for different limits states. The latter aims to evaluate the seismic demand on buildings and structural members.

2. Design strategies

2.1. Performance Based seismic design & limit states

The seismic performance-based design (PBD) (Naeim, 2010, Ghobarah, 2001) allows a control of the structure's vulnerability and economic losses during the structure's lifetime for different earthquake levels which, are related to the limit states defined through specific demand parameters (e.g. inter-storey drift, storey acceleration or the deformation of the structural elements). Design codes follow the PBD applying the Force-Based method by performing the strength capacity verifications for the ultimate limit states, whereas the serviceability limit states are verified by limiting inter-storey drifts. However, the values of inter-storey drift limits reported in design codes seem to be more appropriate for low- to mid-rise timber buildings rather than taller buildings since the deformation contribution due to the rigid body displacement may be significant at the upper levels (Willford et al., 2008).

2.2. Force vs Displacement Based-Design

The performance and the limit states could be achieved by following different design methodologies such as Force-Based Design (FBD) or Displacement-Based Design (DBD) methods (Loss et al., 2018). FBD is widely used by applying equivalent lateral force (ELF) method or with the modal response spectrum (MRS) method (Follesa et al., 2013, Seim et al., 2014). DBD is mainly used in research (Rinaldin et al., 2017, Sustersic et al., 2016). However, the application of DBD methods for timber building is in an early stage (mainly for Direct-DBD and Modal-DBD) and future investigations are needed for taller and hybrid buildings (Loss et al., 2018).

2.3. Capacity design

The capacity design is a seismic design approach used to promote a dissipative behaviour of a structure. The method consists of designing brittle mechanisms stronger than the ductile mechanisms to achieve a ductile failure. Since timber elements are characterized by brittle behaviour, the energy dissipative behaviour is concentrated in mechanical connections. The connections should ensure adequate ductility and low-cycle fatigue strength (Casagrande et al., 2020). The capacity design procedures should be satisfied at the local (connections, timber elements) (Ottenhaus et al., 2021) and global (entire building) level too. Several studies focused on the capacity design approach of mechanical connections (Jorissen & Fragiacomo, 2011, Schick et al., 2013, Vogt et al., 2014, whereas Casagrande et al., 2019 investigated the capacity design at the global level on multi-storey timber buildings (CLT and LFT). However, no specific rules are available for taller timber buildings. Codes do not define how to apply the methodology and which are the dissipative/brittle mechanisms for low-rise buildings (Chen & Chui, 2017).

3. Seismic analysis

3.1. Dynamic properties & Fundamental Period

In seismic design, the dynamic properties (natural frequency, mode shape, damping and linear/non-linear behaviour) allow a correct prediction of the seismic demand and the dynamic response. A numerical comparison between CLT and light-frame structural types (Edskär & Lidelöw, 2019) shows that both systems provided similar natural frequencies due to a balance of stiffness and mass (CLT have higher mass and stiffness than the frame system).

3.2. Seismic and wind loads

Due to the flexibility of taller timber buildings (Foster & Reynolds, 2018), the main design action of the lateral-resisting system could be wind load instead of seismic load as shown in several researches (Tesfamariam, 2022, Shaligram & Parikh, 2018, Chen & Chui, 2017, Tesfamariam et al., 2019). The design action is a function of the dynamic and geometric properties of the whole building and the seismic hazard site. The seismic action is calculated by estimating the design acceleration (accounting for the dissipative behaviour) and the total mass. Theoretically,the seismic action could be limited by reducing the seismic mass and increasing the dissipative and damping capacities. In practice, the possibilities for influencing seismic loads are unfortunately usually limited. The mass is determined by different structural and non-structural factors. Dissipation and damping capacity can have a significant influence on the complexity and cost of the structural connection.

3.3. Finite element modelling strategies: structural elements and connections

Finite element (FE) modelling is a fundamental tool to predict the structural response of a building. For Cross-Laminated Timber (CLT) structures, there are two main approaches to schematize CLT panels: Frame model (Mestar et al., 2020) and 2D orthotropic shells model (Rinaldi et al., 2021). Generally, mechanical connections govern the lateral response of timber structures and are modelled in FE analyses by using link elements or springs depending on the type of seismic analysis and the constitutive law implemented. In the case of taller buildings, the biaxial behaviour of connections should be considered, (D’Arenzo et al., 2021). In particular, it is possible to adopt a biaxial behaviour with a quadratic interaction relationship between tensile and shear loads (Izzi et al., 2018).

3.4. The role of floor diaphragms

The flexibility of floor diaphragms is an important property of timber structures that should be carefully considered during the design stage since it affects the load distribution between shear walls, bracing or cores, as well as adding to local deflections inter-storey drift (Moroder et al., 2014). The flexibility of the floor diaphragm is generally ruled by the slip between panels, which compose this structural element. Hence, the stiffness of the connections used to join together the different panels represent the key influencing parameters for the flexibility of the floor diaphragm (D’Arenzo et al., 2019).

3.5. Interaction of structural elements

The three-dimensional behaviour of the building can influence the lateral response of shear walls. In this respect, the perpendicular walls play an important role, yet typically, they are not considered in practical design. The effects of the perpendicular walls were found in several studies and also in several experimental campaigns on full-size LFT platform buildings to be significant (Van De Lindt et al., 2010, Tomasi et al., 2015). Additionally, the stiffness of the wall-to-floor connections between floor diaphragm and shear walls strongly influences the behaviour of multi-panel CLT shear walls (Tamagnone et al., 2020).

WG3.SG1.05

Standards, Codes and Guidelines

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1. Introduction

Regarding tall wood buildings, there has been a growing interest in construction above eight stories tall over the past 10 years, such as the 18-storey building "Mjøstårnet" in Norway and the 24-storey building "HoHo Wien" in Austria. This report summarises research on standards, codes and guidelines for tall timber building seismic design. Additionally, the limitations are highlighted for further investigations.

2. State of the art

2.1. Standards and guidelines

Eurocode 8 (EC8) was developed to establish harmonised technical rules and to achieve uniform levels of safety in the field of seismic design of structures. The current version of EC8 provides only six pages of specific rules for new timber buildings in Section 8 of EN 1998-1. Due to the growing use of timber in the construction sector, the provisions for timber buildings in EN 1998-1 were markedly extended and a dedicated chapter was introduced in EN 1998-3. The main updates of EN 1998-1 (now prEN 1998-1-2) include the introduction of new wood-based panels such as CLT, revised definitions of the structural types according to the behaviour under seismic actions, a new safety format for seismic verifications, a new definition of the behaviour factor q, new ductility rules for dissipative zones, capacity design provisions and overstrength factors, and detailing rules for all structural types. Other standards and guidelines around the world are also listed in Table 1.

Table 1: Standards and guidelines for seismic design of tall timber buildings

Standards and guidelines	Region	Specification and major updates
the International Building Code (IBC, 2021)	US	1. New construction types (IV-A, IV-B, and IV-C). 2. It allows mass timber buildings up to 18 stories, and greater allowable areas.
National Building Code of Canada, (National Research Council Canada, 2020)	Canada	1. Mass timber gravity system can be built up to 12 stories. 2. The lateral load resisting system (LLRS) system can consist of platform framed CLT shear walls up to 30m height in low seismic areas and 20m height in high seismic areas. 3. The design provisions for this LLRS are provided in CSA O86 (2019). 4. Some guidance is provided in the Technical Guide for the Design and Construction of Tall Wood Buildings in Canada (FPInnovations, 2020)
CNR DT 206-R1/2018	Italy	1. It includes capacity design rules necessary for achieving a dissipative behaviour. 2. Two ductility classes (high and medium) are allowed. 3. Behaviour factors values are specified as a function of the building typology. 4. No limit on the number of storeys.
NZ Seismic Design Guide (WPMA)	New Zealand	It summarises the seismic design rules in the forthcoming timber standard AS/NZS 1720.1 such as capacity design and overstrength.

2.2. Performance-based design methodologies

The rapid growing of tall timber buildings has not been accompanied by immediate standards updating. In addition, performance-based design (PBD), different from the force-based design (FBD) in current design codes, may provide a more proper assessment for the seismic design of timber buildings. Thus, PBD for timber structures as listed in Table 2 has been developed as an alternative.

Table 2: Performance-based design of timber buildings

LLRS system	Research
Wood frame buildings	Pang and Rosowsky (2009)
Glulam timber frame buildings	Zonta et al. (2011)
Timber-steel hybrid structures	Di Cesare et al. (2019) Teweldebrhan and Tesfamariam (2022)

In the PBD, the definitions of yield points and ultimate failure points were found to be inconsistent among different mass timber LLRS systems, which makes it difficult to compare the results from different tests, such as the ductility of structures.

2.3. Force reduction factor for FBD, and safety factors

To reflect the energy dissipation, the ductility characteristics, and the dependable portion of reserve strength in the LLRS of the buildings, the use of the force reduction factor (FRF) is specified by most building codes for the FBD as an alternative to full dynamic analysis. It is also called behaviour factor q, response-modification factor (RMF) and force modification factor (FMF) in Eurocode, US codes and Canadian codes, respectively. Typically, the FRFs are given for each LLRS and diverse values are provided by different design codes. Table 3 lists research on FRFs for multi-storey timber-related systems.

Table 3: Research summary on force reduction factor (FRF)

LLRS system	Research	Results
CLT buildings with light-gauge hold downs	Ceccotti et al. (2016) Sustersic et al. (2016)	q factor = 2 and 3 for single monolitic and coupled (segmented) CLT shear walls q factor = 2 and 3 for buildings made with monolithic and segmented CLT walls
	van de Lindt et al. (2022)	RMF=3.0 for CLT shear wall systems with 2:1 or mixed aspect ratio panels up to 4:1; RMF = 4.0 for CLT shear wall systems made up of only 4:1 aspect ratio panels (following the FEMA P-695 procedures)
Timber-steel hybrid structures	Zhang et al. (2015)	ductility factor = 5.0
	Chen and Chui (2017)	FMF = 2
	Khajehpour et al. (2021)	FEF = 6.0 (following the FEMA P-695 procedures)
Timber-concrete hybrid structures	Tesfamariam et al. (2021)	FEF = 3.0 (following the FEMA P-695 procedures)
Post-tensioned timber structures	Sarti et al. (2017)	RMF = 7.0
	Pei et al. (2019)	RMF = 6.0

Eurocode 5 provides safety factors for solid timber, engineered wood products, plates, and connections. The revision of Eurocode 5 will also provide the partial safety factors of new engineered wood products such as CLT. Eurocode 8 refers to Eurocode 5 for the partial safety factors to be used in seismic design of non-dissipative structures, whereas the partial safety factors for accidental design situations (generally unitary) are prescribed for dissipative structures.

3. Conclusions

Although significant research efforts have been put into the standardisation of mass timber LLRS, some challenges remain and require further investigations.

1. Although a complete revision of the timber section of Eurocode 8 is in place, some issues such as the design of hybrid buildings with different LLRS at the same levels or superimposed still have to be resolved.
2. Other methodologies such as PBD can be a complement to current standards, but consistent definitions are still required for the wide application.
3. Although innovative mass timber LLRS showed enhanced structural performance through testing, design parameters of these LLRS such as FEF and safety factors have not been verified by codes through a rigorous procedure (e.g. FEMA P695). In addition, the parameters are usually code-dependent, so the derived values are not directly transferable to different codes.

WG3.SG1.06

Case Studies

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1. Introduction

Tall Timber Buildings (TTBs) are becoming increasingly common around the world (Krötsch & Müller, 2018, Kuzman & Sandberg, 2016, Kuzmanovska et al., 2018), and their seismic design presents new challenges to structural engineers (Demirci et al., 2019, Stepinac et al., 2020, Moroder, 2016). Therefore, this section is dedicated to the analysis of several existing TTBs (based on idea of Salvadori, 2021), selected as Case Studies.

A comprehensive overview can be found also in Svatoš-Ražnjević et al. (2022), who examined the range of typologies and morphologies in current multi-storey timber construction in terms of structural and material aspects. The main objective of this contribution is to find and compare different features of TTBs (i.e., seismic force resisting systems, design strategy, and analysis models) developed by structural engineers to overcome design (calculation) or execution issues on seismically prone areas.

This classification and comparison could prove useful in formulating a handbook or standards section for TTBs. To achieve a simple but exhaustive analysis for each TTB case study, a data sheet template was designed. The latter is divided into different sections dealing with the main seismic design parameters. A complete register was prepared for each case study, but in the following (two-page State-of-The-Art Report) only some relevant aspects are discussed in depth: the location, the height/tallness, the lateral force resisting system (LFRS), the design strategy, and the building analysis model.

For the most significant parameter, the related part of the data sheet is given, where some values are shown as examples (in this STAR, the data sheet of Viale Giannotti in Florence is given). The complete data sheets and the description of the parameters are included in the Annex A.

2. Parameters and Information reported in Data Sheet

2.1. Location and Seismic Data

Location and Seismic data are provided in order to identify site conditions and seismic action so that researchers and designers can immediately understand the seismic demand and whether they are relevant to the design process (Table 1). In addition, the behaviour factor (q-factor) used in the analysis of the building is also provided.

2.2. Height and Tallness

Criteria need to be established for classifying a building into low-rise, mid-rise, and high-rise subcategories, for example, depending on the number of floors (Foster et al., 2018). Standardization of the various classification methods described in the literature and clear definition of each subcategory is essential to provide an overview of the existing TTB. Tallness and height are generally not the same thing. Height is objective; it is a measurable property of a physical object. On the other hand, tallness is subjective; it is a description of a physical object that implies some form of contextual reference (Foster et al., 2016).

2.3. Lateral Force Resisting System (LFRS)

The LFRS can be divided into hybrid and timber-only systems. The hybrid systems, especially where lateral forces are transferred to steel or concrete structures (cores), can be designed and built with current knowledge, while the timber-only systems require special high-performance connections and new design methods (Moroder, 2016).

Table 1. Information in the fact sheets on the lateral force resisting system.

7. Lateral Force Resisting System (LFRS)					
Reinforced concrete slab and wall (Basement) Floors 1 to 6: 5-layer spruce/pine CLT wall panel (100mm to 180mm thick) ETA-14/0349 5-layer spruce/pine CLT slab panel (140mm to 180mm thick) ETA-14/0349					
7.1. Cores	n.	-	Yes		~~No~~
			Side	Center	
7.2. Connections					
All CLT elements are connected to each other and to the foundation by use of steel plates and screws.					

2.4. Seismic Design Criteria

The Standards used for the design and verification of the building are given to explain the calculation methods, loads, safety and over-strength factors, etc. used by the designers. Similar buildings may have significant differences due to the Standards adopted.

Table 2. Information in the fact sheets on the seismic design criteria.

9. Seismic Design Criteria		
9.1. *Reference Standards*		
Eurocode 1995-1-1; Eurocode 1998-1-1; Italian National Standard (NTC 2018)		
9.2. *Capacity design*	~~Yes~~	No
9.3. Coincidence of the center of rigidity and the center of gravity	~~Yes~~	No

2.5. Analysis model

It is important to understand how the designers modelled the buildings and what modelling strategy they used for timber elements, connections, and other structural elements of the building. Project analyses could provide insight into how designers resolved critical issues and what technical solutions they chose. From the analyses of some case studies, it is possible to define the lateral seismic loads acting on TTB and especially internal forces on connection systems (as a function of location, number of floors, LFRS, dynamic features avoiding/reducing resonance response of TTB).

Table 3. Details in the fact sheets on the building analysis model.

10. Analysis Model		
10.1.	***Model Description***	FEM:2D shell elements (mesh of 0.5mx0.5m)
10.2.	***Analysis Software***	SAP2000
10.3.	***Seismic Analysis***	Modal analysis

WG3.SG1.07

Interactions and Conflicts in Holistic Design

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1. Introduction

Building design is a complex, multidisciplinary engineering activity that involves making difficult trade-offs to balance competing objectives such as safety, reliability, performance, and cost. The design of a typical building involves the collaboration of many different disciplines – architecture, civil engineering, building services, etc. – for a relatively short period of time compared to the operational phase of the building. Each designer makes decisions based on design requirements, constraints, and inputs from other disciplines. Due to the fragmented nature of knowledge, no single professional has all the knowledge needed to design a complex facility. Although many building designers still work in parallel and independently, using different engineering tools, the benefits of collaboration are widely recognised (Fruchter, 1999, Lattke & Hernandez-Maestschl, 2016, Leoto & Lizarralde, 2019, Santana-Sosa & Fadai, 2019) including optimising functions, minimising costs and reducing errors.

Good holistic design cannot be achieved by focusing on a single design segment, as each issue can pull the project in a direction that may appear regressive to the other segment. It is often argued that integrated design is an effective way to improve collaboration in construction projects. Unlike the traditional silo-type and linear design process, integrated design is based on early stakeholder engagement and a holistic approach to project decision making. When designing a building with timber, the specific transfer of know-how between timber specialists or contractors and structural engineers or architects should take place earlier than for other structures, especially if they do not have previous and extended experience with timber construction. Ideally, on-site assembly strategies and off-site production systems should be considered at the design stage, allowing for an efficient workflow later on. Designing timber structures, therefore requires a more detailed approach and knowledge concerning, among others, material properties and structural behaviour, since all requirements related to fire, sound, moisture and thermal protection are integrated into the structural layer. At the same time, all disciplines, i.e., architectural, structural, mechanical, electrical, and plumbing (MEP), need to be coordinated and integrated in the design phase, as openings for MEP should be created at the factory and changes at the construction site should be avoided.

Current approaches to collaboration in practice often focus on integrating and managing multiple models from multiple designers. Building Information Modelling (BIM) plays an important role in facilitating collaboration. BIM provides the ability to electronically model and manage the large amount of information contained in a construction project, from conception to completion. In the construction design process, changes and modifications are inevitable even in the contemporary BIM approach, where it needs to be ensured that designers have an up-to-date version of the model.

2. Interactions and conflicts

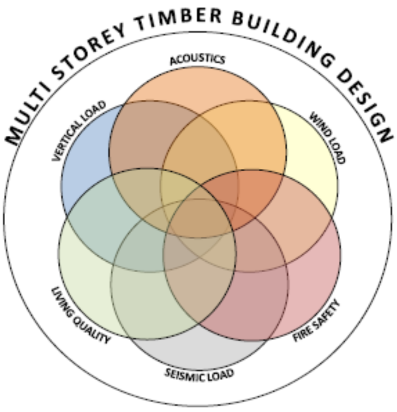
The collaboration and interaction of the various stakeholders (i.e., architects, engineers, contractors etc.) in the design of large timber buildings is often not clearly structured. Practical experience with prefabricated timber buildings shows that specific knowledge of production planning and methods for off-site timber construction comes too late, resulting in changes, problem solving, pressure and additional labour that are usually unpaid (Lattke & Hernandez-

Maestschl, 2016). Therefore, the design of taller timber buildings should be done with intensive collaboration between the members of the design team in the early stage (Aberger et al., 2018).

A fundamental conflict arises from the requirement to achieve structural safety with an optimized (cost-effective) design. Structural optimization based on cost efficiency may conflict with end-user comfort (Stepinac et al., 2020). One such example is the sound insulation of flanking noise and the lateral stability of timber structures. The principles of sound insulation currently applied to timber buildings are in complete contradiction with the design requirements resulting from wind or earthquake loads (Azinović et al., 2021). The acoustic demands decouple elements to limit the transfer of noise, while earthquake & wind requirements tie them together rigidly to resist lateral loads. Another example is the wind serviceability design where a higher mass is needed to reduce building vibrations (Malo et al., 2016). In some cases, therefore, additional mass is intentionally added to the structure. However, this mass also potentially increases seismic forces. There are also collisions between the desire to keep timber visible for human comfort and to meet fire safety and acoustic requirements (Buchanan, 2016).

However, conflicts in timber buildings can also arise due to different structural requirements, such as seismic design and robustness. Both focus on capacity based design and require connector ductility, but within different connection lines. Seismic design also needs to resist many load reversal cycles, whereas this is not essential for robustness.

The list of different type of conflicts can be continued, and it becomes even more challenging when you considering all the phases that a building goes through: design, construction, use, and finally demolition and recycling (Sandin et al., 2022), which is an important part of how sustainable buildings need to be designed for the future (Campbell, 2018). Šušteršič et al. (2021) summarized the possible conflicts that can arise in the multi storey building design (Figure 1) that effect both the load resisting and serviceability criteria of a building.



INTERACTIONS + positive - negative N neutral	VERTICAL LOAD	WIND LOAD	SEISMIC LOAD	FIRE SAFETY	ACOUSTICS	LIVING QUALITY
VERTICAL LOAD		+	-	-	+	N
WIND LOAD	+		+	N	-	N
SEISMIC LOAD	-	+		N	-	N
FIRE SAFETY	-	N	N		+	-
ACOUSTICS	+	-	-	+		-
LIVING QUALITY	N	N	N	-	-	

Figure 1: Interaction of a few different building design fields and their inherent collisions, either positive or negative, that need to be resolved for multi-storey taller timber buildings [from Cost Action CA20139 MoU].

3. Conclusions

Since generally proven solutions to such conflicts in design are not available in codes or standards, it is usually up to the design teams – assuming they have the appropriate knowledge and experience – to find solutions that can at least partially satisfy all parties. A holistic design must be guided by the technical requirements of the various disciplines, which should be weighted and hierarchized, taking into account the various interactions that result from a variety of objectives such as safety, reliability, performance, and cost. In practice, a designer must rely on his ability to make decisions based on calculations, but it is up to his talent to have a holistic approach to the requirements that ensure better design, construction and use.

WG3.SG1.08

Comments and Questions COST Action Helen Meeting Gothenburg

This chapter lists the questions asked and comments given during the COST Action Helen meeting in Gothenburg on the 4th and 5th of October 2022.

1. Questions

May the performances of timber connections subjected to cyclic loads change due to durability issues and/or additional fire loads?

Answer during event: majority of research on the cyclic behaviour of timber connections has been primarily focused on new elements subjected to cyclic loads. Recent studies have shown that the mechanical performances of the connections are influenced by thermal sources, and this is expected to be an issue for cyclic performances as well. Limited information is available on how durability issues (such as moisture content) may influence the cyclic performances of the connections.

How adhesive connections behave under seismic loads?

Answer during event: Adhesive connections have generally high mechanical performances, however, characterized by brittle failure mechanisms. For this reason, in seismic design analyses, they are often capacity designed and their post-elastic behaviour is not exploited.

Why different inter-storey drifts should be suggested for tall and mid-rise buildings?

Answer during event: Inters-torey drifts are generally used in Performance Based Design (PBD) to control the damage of structural and non-structural components of buildings subjected to low intensity earthquakes, and thus for serviceability limit state (SLS) design. The inter-storey drift is the sum of two contributions: i) displacement due to the cumulative rotation, which does not induce any damage, and ii) racking deformation, which induce damage. While in mid-rise buildings the inter-storey drift is mainly due to the racking deformation of the resisting system, in tall buildings the inter-storey drift accounts both displacements due to the cumulative rotation and racking deformations. Due to the “additional” displacement given from to the cumulative rotation of tall buildings, which does not induce damages, different inter-storey drift limits should be considered for tall timber buildings.

2. Comment

A member indicated that first topic of SG1 on *Seismic Load Resisting Systems* may have interactions with WG1.

Part 2

Fire

Sub Group (SG) 2

WG3.SG2.01

Building Regulations

Daniel Brandon, RISE (Sweden); **David Barber**, Arup (Australia)

1. Suitability of building regulations for taller mass timber buildings

Regulations to ensure fire safety in buildings differ vastly among countries. The most extensive global overview of fire safety regulations for buildings with timber structures is given by Östman (2021). The overview focusses on regulations regarding (1) the potential for lining materials to contribute to the development of a fire (for example by reaction-to-fire classes), and (2) fire resistance. However, in contrast to other combustible lining materials, mass timber is relatively thick and can contribute to the fuel of the fire for periods long after the development phase. This contribution can be increased by glue line integrity failure (i.e. fall-off of glued lamellas), fall-off of fire protection and significant charring behind fire protection (LaMalva and Hopkin, 2021). The fuel contributions of mass timber can lead to prolonged fires, which reduces the likelihood of the building to survive burnout scenarios (in which fire suppression and sprinkler suppression are unsuccessful).

Even though many building regulations do not explicitly require a building to survive burnout scenarios, analysis of background documents identified that the ability of conventional structures (with limited combustible materials) to withstand burnout was, at least in some countries, the underlying performance goal of certain fire resistance requirements (Brandon et al. 2022). Another performance goal for taller buildings (usually up to approximately 8 floors) identified in the study was a goal to contain the fire to a limited building part and allow enough time for fire service intervention.

Ensuring a building can withstand burnout cannot be done using reaction-to-fire classes and fire resistance requirements alone. Most building regulations have not been adjusted to meet such performance goals also with mass timber structures. A few exceptions are detailed in the next section.

2. Building regulations adjusted for mass timber structures

The building regulations in only a few countries (e.g. USA, Canada, Australia) have been significantly adjusted for the recent revolution of mass timber structures. These regulations involve different measures to limit or prevent possible effects resulting from the involvement of mass timber, such as increased fire duration and/or fire exposure, smoke development, risk of flashover, external façade exposure. Often additional regulations are in place to limit the extra sensitivity to fire spread through details such as cavities and connections. Requirements regarding automatic water-based fire protection and fire safety during the construction phase are also in place. Table 1 indicates such regulations for the USA, Canada and Australia. The different performance goals vary from restricting the involvement of mass timber (e.g. IBC 2021, type IV-A), to surviving burnout scenarios (e.g. IBC 2021/G147-21, type IV-B & 2020 NBCC), and containing the fire to allow enough time for successful fire service intervention (IBC 2021, type IV-C & NCC 2019 Type B and C).

Table 1: Building regulations in the USA, Canada and Australia

Country	**United States**				**Canada**	**Australia**
Building Code	**IBC 2021**			**G147-21**	**2020 NBCC**	**NCC 2019**
Building class	Type IV A	Type IV B	Type IV C	Type IV B	Encaps. MT	Type B or C
Height	≤18 floors	≤12 floors	≤6, 8, 9 floors	≤12 floors	≤12 floors	<25m
Required Fire resistance (FR)	flr. 120 min roof 90 min **rest 180 min**	Roof 60 min, **rest 120 min**			flr. 120 min	60, 90 or 120 mins, depending on use and height
Required fire protection	max. of - 80 min - 2/3 * FR eg. 3x 15.9 mm Type X, 1"layer on floor	max. of - 80 min - 2/3 * FR. eg. 2x 15.9 mm Type X, 1"layer on floor	Not dependent of construction type	max. van: - 80 min - 2/3 * FR. eg. 2x 15.9 mm Type X, 1"layer on floor	50 min eg. 2 x 12.7mm type X gypsum, 38mm concrete on floor	Above three flr, 30 mins protective covering to all timber (1 x layer 16mm fire rated plasterboard). Top side of floor can be Unprotected
Maximum exposed surface area	0%	20% ceiling or 60% of 2 walls	100%	100% plaf. of 60% van 2 wanden	35% of all walls	Above 3 floors, timber is protected
Distance between exposed surfaces	Not applicable	4.6 m between exposed surfaces	No limitation	4.6 m between exposed surfaces	Exposed surfaces same direction	Only topside floor can be exposed >3 floors
Resist glue failure	Demonstrated to pass tests of PRG 320 (2019) Annex B & 6.2.1-b					Not addressed
Limitation of fire development	NFPA 13 sprinklers >36m, 2nd water supply & 100% encapsulation	NFPA 13 sprinklers >36m, 2nd water supply & 100% encapsulation	NFPA 13 sprinklers	NFPA 13 sprinklers >36m, 2nd water supply & partial encapsulation	Sprinklers & partial encapsulation	Fully sprinkler protected building & >3 flr, all surfaces protective covering
Limitation fire spread through intersections	Fire sealed with ASTM C920 sealant, or ASTM D3498 adhesive				Independent of meterial	16mm fire rated plasterboard protection to MT surfaces
Limitation fire spread through cavities	No exposed wood in hidden voids, non-combustible protection layer is required				· fire stops · cavity ≤ 20 m	Fire stops for light frame
extinguishing facilities	· >36m, 2nd water supply · > 4 flr., stand pipe				> 3 floor., Standpipe	Sprinkler protection throughout building
Fire Safety during construction	· Stand pipes and water supply · >6 flr.: gypsum on all floors except for the 4 upper floors before erecting new floors · hydrants with additional water flow requirements				Hydrants with specific waterflow	No specific additional requirements
Facade requirements	Non-combustible & 40 min protection between façade and mass timber, for example . 15.9 mm type X gypsum				≤6 flr, ≥90% non-comb. lining or ULC-S134 tested	No additional lilmits for timber buildings. Combustibility restricted above 3 floors

WG3.SG2.02

Real fire exposure

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1. Introduction

In comparison to common non-combustible high-rise buildings, when timber is used as a structural material it can significantly alter the compartment fire dynamics. This section aims to describe the real fire behaviour in compartments with (partially) exposed timber (e.g., engineered wood products (EWP)), and protected timber elements that may contribute to the fire if their encapsulation fails. Experimental findings to date are summarised, and design challenges are highlighted for the growth phase, fully-developed (post-flashover) phase, and cooling phase of a realistic or 'natural' fire.

2. Thermal degradation of timber

Depending on the specific species, wood is generally composed of cellulose (40-55%), hemicellulose (15-35%), lignin (18-40%) and other minerals and extracts (Neugebauer 2019). As other construction materials, wood is subjected to both physical and chemical changes at elevated temperatures. At temperatures above 100°C, the water and moisture within wood move, diffuse and/or evaporate. Wood typically starts decomposing and releasing combustible gases for temperatures above 200°C, known as pyrolysis. Above 280-300°C, residual carbonaceous char is formed upon pyrolysis, and in the presence of oxygen, the wooden char further degrades in oxidation reactions, typically in the temperature range 400-500°C (Browne 1958). Accordingly, the thermal decomposition of wood alters its mechanical properties. Compared to ambient temperature conditions, wood stiffness and strength reduces for relatively low temperatures, and the wood load-bearing capacity is considered null above charring temperature, assumed equal to 300°C (EN 1995-1-2:2004).

3. Fire growth and available safe egress time

The time to untenable conditions in a compartment can be shortened if timber becomes involved early in the fire. This regards the development of compartment temperatures (Li et al., 2015; Su et al., 2018a), and can also include toxicity (Hull et al., 2016) and visibility. Flashover in exposed timber compartments has been observed to occur earlier than in non-combustible compartments (Li et al., 2015; Su et al., 2018b), exacerbated by the area, number and position of exposed surfaces. Exposed timber ceilings in large open-plan compartments can promote much faster fire spread rates and transition to fully-developed fire conditions once the combustible ceiling ignites (Gupta et al., 2021; Kotsovinos et al., 2022; Nothard et al., 2022).

Active fire protection systems can in many regulatory frameworks be used as a trade-off to reduce other fire safety provisions, such as the fire resistance of structural elements or compartmentation. In practice, sprinklers are often provisioned to protect taller buildings built in timber to control any occurring fire and prevent fire growth and spread (Östman et al., 2022), and hence effectively lower the risk of untenable criteria (Frangi and Fontana, 2005; Zelinka et al., 2018). Experiments with exposed CLT surfaces show that the HRR from a compartment fire can be significantly reduced through sprinkler systems compared to non-sprinklered scenarios (Zelinka et al., 2018). However, sprinklers are not 100 % effective, and potential failures to activate the system or control the fire early enough can lead to total suspension of their operation, in which case a structure would have the same protection as with no sprinklers present. Therefore, the use of sprinklers to relax other safety provisions must follow a well-reasoned risk assessment to avoid compromising building safety.

4. Fully-developed fire dynamics

A fire may be defined as fully-developed once it has reached a quasi-steady peak state in which its size is constrained by either the availability of oxygen (under-ventilated compartments) or by the availability and configuration of fuel (well-ventilated compartments) (Thomas et al., 1967; Torero et al., 2014).

During the fully-developed phase, the additional fuel load from burning timber elements will increase the total heat release rate (HRR) of the fire (Crielaard et al., 2019; Gorska et al., 2020; Hadden et al., 2017; Li et al., 2015; Su et al., 2018a). In under-ventilated compartments, this excess fuel will combust outside of the compartment, with little impact on the amount of heat generated internally. In well-ventilated compartments, ignition of wooden surfaces will increase the internal HRR, increasing temperatures and heat fluxes within the compartment (McNamee et al., 2021), and potentially causing a travelling fire to transition to flashover (Gupta et al., 2021; Kotsovinos et al., 2022; Nothard et al., 2022).

In either case, the configuration of exposed surfaces may alter the flow fields within the compartment, resulting in highly non-uniform temperatures or heat flux distributions (Gorska et al., 2020; Pope et al., 2021).

Exposed timber has also better insulative properties compared to concrete or masonry, resulting in higher heat accumulation and possibly higher fire temperatures in compartments (Bartlett et al., 2020; Lange et al., 2020; Węgrzyński et al., 2020).

Burning timber surfaces may also extend the duration of the fully-developed period significantly if self-extinction of the timber surfaces is not ensured. Char fall-off and delamination, failure of encapsulation, or excessive heat feedback may all result in continued burning until failure of the compartment (Bartlett, 2018; Hadden et al., 2017; McGregor, 2013; McNamee et al., 2021; Medina Hevia, 2015; Pope et al., 2021).

The duration of the heating and cooling phases, and the progression of the thermal wave beneath the char layer (Gernay, 2021; Wiesner et al., 2021a), define the structural performance of timber elements under real fire conditions.

In engineering standards, the char layer depth is specified by *standard fire* exposure and pre-defined charring rates (CEN, 2004). Considering variable fire durations and severity in reality, actual charring rates often differ from such values.

4.1. Exposed wood – Fire induced delamination and char fall-off

When some EWPs are exposed to fire conditions, the adherent (timber) and the adhesive do not deteriorate at the same rate, which can lead to unpredictable failure modes, both in timber and the bond line region (Emberley et al., 2017b; Frangi et al., 2004), seen most prevalently in CLT.

Such failure modes for CLT are (i) char fall-off and (ii) fire induced delamination. Char fall-off appears in any part of the charred cross section. Fire induced delamination, also known as glue/bond line integrity failure (Hopkin et al., 2022), appears in the proximity of the bond line at temperatures lower than the timber pyrolysis temperatures. Both failures initiate a change in fire dynamics and heat transfer (Bartlett et al., 2017; Emberley et al., 2017c; Frangi et al., 2009;

Hadden et al., 2017; Su et al., 2018a), but delamination also influences the loss of structural capacity and fire dynamics (Wiesner et al., 2021b, 2020).

Both failures can be influenced to a limited degree by edge bonding and by increasing the thickness of the first lamella. Different types of adhesives also exhibit varying delamination and char fall-off behaviours and can improve performance (Brandon and Dagenais, 2018; Čolic et al., 2021; Čolić, 2021; Crielaard et al., 2019; Frangi et al., 2004; Hopkin et al., 2022).

4.2. Encapsulated wood – Failure of encapsulation

Failure of encapsulation is defined by either a loss of mechanical integrity or inadequate thermal insulation (Xu et al., 2022) and is usually one or a combination of (i) insufficient thickness of the encapsulation materials, (ii) insufficiently covered compartment area (i.e., fuel load is increased by exposing the timber surfaces), or (iii) mechanical failure of the encapsulation joints and fasteners (Chorlton et al., 2021).

Fire resistance of plasterboards is currently tested under the standard fire curve, but they are known to be sensitive to the heating rate, and fail faster in most fully-developed compartment fires (Brandon, 2018). Charring of wood behind encapsulation boards is allowed based on fire resistance requirements alone. However, in real fires, significant charring due to smouldering behind insulating boards is likely not to stop automatically even after the compartment has cooled down, and can lead to char depths greater than those of exposed timber in the long run (Su et al., 2018a).

A challenging aspect is that the failure of encapsulation during the cooling phase may lead to continued burning, or re-ignition of the timber structure and possible subsequent re-emergence of a fully developed fire stage (Su et al., 2018a, 2018b).

Intumescent coatings can also be used as a passive fire protection of timber elements while maintaining their aesthetic value. However, effectiveness can vary greatly depending on the heating conditions, and small differences in applied thicknesses may have large effects on the resulting protection (Hartl et al., 2020; Hasburgh et al., 2016; Lucherini et al., 2019).

4.3. Exposure to facades and external objects

The additional combustion energy of mass timber that is involved in a fire can lead to increased exposure to facades and external objects, especially in ventilation controlled fires (Brandon and Östman, 2016; Frangi and Fontana, 2005; Hakkarainen, 2002).

The flame height and thermal exposure to the facade is, among other things, dependent on the surface area of exposed timber, and the ventilation opening dimensions, (Bartlett and Law, 2020; Gorska, 2020; Sjöström et al., 2021) and have been performed with façade measurements for statistically slender and small openings to get exposures on the severe end of the spectrum. Models predicting flame height are published by Hopkin and Spearpoint (2021) and Gorska (2020) (Gorska, 2020; Hopkin and Spearpoint, 2021). Many national standard façade fire tests, do not induce the same level and duration of exposure as can be expected in bad-case fire scenarios (Sjöström et al., 2021).

Currently, fires jumping from floor to floor in timber buildings can be prevented by severe architectural constraints (Law and Kanellopoulos, 2020) or fire resistance glazing, which is more significant if exposed timber is present.

4.4. Time-equivalency (use of classification fire resistance and k classes for PBD)

Time-equivalency methods are needed to enable the use of fire resistance ratings of products (e.g., fire separation walls, fire doors, fire sealants, penetrations) in a performance-based design approach, where it is infeasible to explicitly quantify the performance of all building elements under real fire exposure (Wade, 2019). “Equivalent time” indicates the duration of fire resistance testing required to induce the same structural damage as a defined duration of a specific design fire or real fire. However, ensuring a fire will fully stop (including residual smouldering) may require the involvement of fire-fighting services in the fire strategy, or full encapsulation of the timber structure. Such methods should account for the fuel contribution of the timber structure, which depends on the real compartment fire dynamics.

For timber structures, due to high uncertainties caused by knowledge gaps, experimental validation of such methods or extensive data derived from such experiments is needed, for accurate assessment through performance-based design.

5. Cooling phase fire dynamics

Once the moveable fuel load in a compartment has been consumed, the fire may decay sufficiently to allow the burning timber elements to decay through to flaming self-extinguish and burnout (Bartlett et al., 2017; Brandon et al., 2020; Crielaard et al., 2019; Emberley et al., 2017d; McGregor, 2013; Medina Hevia, 2015; Zelinka et al., 2018). If flaming self-extinguishment is achieved, the compartment will continue to cool rapidly in the gas-phase, but the effects of solid-phase heat-transfer and smouldering may be apparent over much longer timescales (Kotsovinos et al., 2022; Wiesner et al., 2022).

In contrast to compartments with non-combustible boundaries, timber compartments might never achieve ‘burnout’ under certain conditions, and may continue to burn long after the moveable fuel is consumed, depending on the area of exposed timber and the type of timber, such as CLT that delaminates. Burnout of timber compartments is considered to occur when flaming extinction (extinguishment) or glowing and smouldering extinction occurs. However, there is no general consensus on the definition of burnout (Schmid et al., 2021).

In any case, ensuring the conditions for flaming self-extinguishment of exposed timber surfaces is one step in designing for the eventual burnout of a compartment fire. This can be achieved when the external radiant heat flux onto the surface of the charring timber falls below a critical value that ranges between 30-50 kW/m^2 (Bartlett et al., 2017; Cuevas et al., 2020; Emberley et al., 2017d, 2017a; Terrei et al., 2021). Thermal degradation of the timber impacting strength will still occur after flaming self-extinguishment and needs to be accounted for.

The charred timber acts as a thermal insulator, and the critical heat flux criteria applies once the char layer thickness has reached a quasi-steady state, so it can be compromised by char fall-off or delamination that increases the heat transfer to the underlying timber (Bartlett et al., 2017; Emberley et al., 2017b; Medina Hevia, 2015; Su et al., 2018a). Once the moveable fuel load has been consumed, the residual thermal exposure to the exposed timber will be dominated by radiative and convective heat feedback from the other compartment surfaces. Consequently, the area and configuration of exposed timber surfaces must allow for the residual heat fluxes in the compartment to fall below the critical value for flaming self-extinguishment, noting that thermal degradation will still occur. This also requires that the integrity of encapsulation is maintained where installed (Gorska et al., 2020; Hadden et al., 2017; Medina Hevia, 2015; Su et al., 2018a; Xu et al., 2022).

The configuration of exposed timber surfaces may have a greater influence on the potential for flaming self-extinguishment than the total exposed area, since this will control the heat transfer between surfaces. For example, it may be safer to expose a ceiling than a wall, because the burning rate of the ceiling is lower (Brandon et al., 2020; Gorska et al., 2020). Ventilation, and the associated availability of oxygen, also has a significant impact during the decay phase for char oxidation, which can be an important process as the heat flux from combustion of the moveable fuel load decreases (Harmathy, 1978). Localized smouldering may continue for many hours following the cessation of flaming, even leading to failure of compartmentation or structural collapse without intervention (Kotsovinos et al., 2022; Wiesner et al., 2021a). This risk is associated with much longer timescales that allow for fire-fighting detection of smouldering, management, though once detected is relatively easy to extinguish. Therefore, specific fire-fighting strategies are needed for smouldering in concealed spaces.

WG3.SG2.03

High Structural Fire Performance Solutions

Felix Wiesner, The University of British Columbia (Canada); **Daniel Brandon**, RISE (Sweden); **Miriam Kleinhenz**, ETH Zurich (Switezerland); **Andrea Lucherini**, Frisbee & Ghent University (Slovenia); **Chifliganec Cvetanka**, Ss. Cyril and Methodius University in Skopje (North Macedonia); **Alar Just**, Tallinn University of Technology (Estonia); **Marko Veizovic**, University of Belgrade (Serbia)

1. Introduction

Engineered timber has been and continues to be chosen as the main structural material for taller and more ambitious buildings. This development imposes new load bearing demands on timber, both in terms of stress direction and magnitude. Both structural and fire safety engineers must critically evaluate technical issues arising from this development, to ensure levels of safety that are equivalent to those accepted for tall and complex buildings employing legacy materials (i.e. concrete and steel) for their structure. This section introduces and details the most pressing concerns that arise from a structural perspective for mass engineered timber buildings in case of fire.

2. Fire protection by non-combustible boards

Most research on passive fire protection cladding to timber is performed for gypsum boards. A comprehensive review of fire resistance tests has been performed by Just *et al.* (2010) to determine empirical relationships of gypsum based board performance for fire resistance calculations. Such a review is lacking for other types of non-combustible cladding. Insulation behind cladding can also contribute to the fire performance but can fall off once the cladding fails. Despite the significant effect failing protection can have on the fire performance, only one study has been found that compares the fire performance several fixing methods for protective cladding (Tiso, 2018).

3. Structural fire performance of timber elements

Taller building heights necessitate the use of engineered timber with increased cross-sectional dimensions for vertical load paths and of long-span, multi-span, and point supported beam and floor systems. One resulting issue is that some large members cannot be tested in existing furnace arrangements, causing a scarcity of data on their fire performance. Consequently, engineers must rely on engineering considerations to scale load bearing capacity for vertical load bearing members in fire. Existing tools for the assessment of load bearing members are either empirical (Li 1977) or use approximate explicit considerations of losses of mechanical properties to assess load bearing capacity during expected fire duration (König, J., & Walleij, L. (2000), Eurocode 5 (2009)).

The fire design of load bearing timber beams and floors can be use either the code prescriptive simplified calculation method (e.g. effective cross-section method from Eurocode 5 (2004)) or through advanced calculation methods which explicitly account for temperature induced loss of mechanical properties. The former has been shown to not be reliable for more complex structural systems, such as CLT, while the latter relies on knowledge of the overall thermal profile. The behaviour of engineered wood systems exposed to fire are also dependent on the type of connection used in the system (e.g. glued or mechanical connections). Elevated internal member temperatures can cause a reduction or loss of composite action due to a reduction in timber strength or adhesive performance (Klippel, 2014, Wiesner *et al.*, 2021a).

Columns and walls are usually subjected to large compression forces and timber mechanical properties are most affected in this mode of loading. As a result, large losses in structural capacity can arise during the cooling phase after a fire and further research is required to ensure that furnace results are appropriately applied to real fire exposure (Gernay, 2021, Wiesner et al., 2022).

4. Connections and penetrations

There a number of literature reviews for connections (Maraveas et al 2013, Audebert et al 2019, Brandon *et al.*, 2019) which indicate that most fire resistance tests of connections between glulam or sawn timber are not directly applicable to taller buildings because of the load direction and a fire resistance of 60 minutes or less, see (Figure 1). Higher fire resistance rated connections for glulam beam to column members are available (Barber 2017).

Frangi, (2001) and Fornather, (2003) indicated that a flow of hot gasses through small gaps can heat up the timber enough to start smouldering at connection and penetration interfaces, which eventually can accelerate burn-through. Recent fire testing has shown the use of intumescing seals can slow or prevent this issue (Barber et al 2022) Multiple studies of full-scale compartment tests indicated that wall-ceiling connections in CLT structures are sensitive to burn-through (Su *et al.*2018, Brandon *et al.* 2021). There is however no fire resistance test standard for such connections.

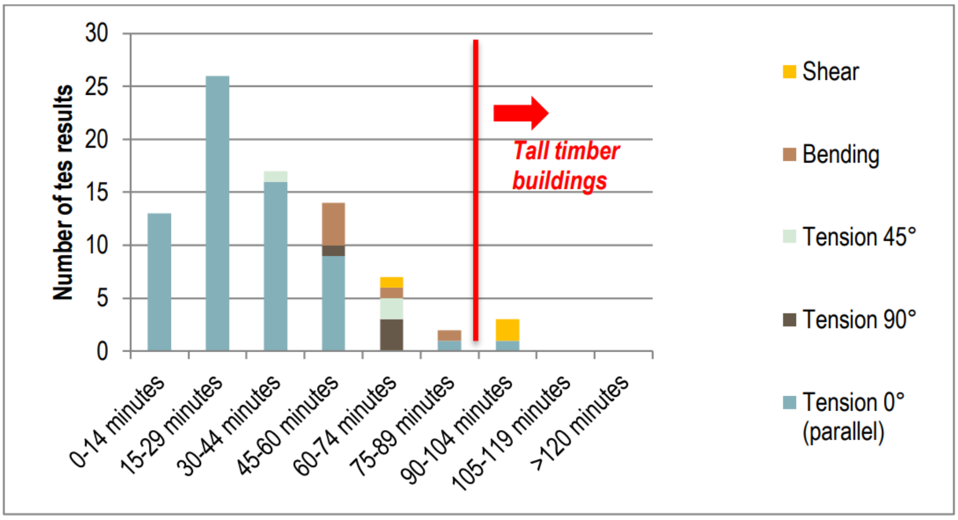

Figure 1: Overview of published fire resistance tests of glulam and sawn timber connections by load direction and fire resistance, from Brandon et al. (2019).

Buildings are separated by compartment walls and floors and penetrations in these fire barriers for electrical, mechanical, plumbing and structure should be minimised. Fire sealing a penetration requires a product tested and approved for use, typically with specific characteristics, level of performance and range of application (Werther et al 2012, Werther et al 2016). Although European fire resistance test standards require penetrations through non-conventional supporting structures such as CLT, only one publication was found that included results of such tests (Ranger *et al.* 2018).

5. Integrity failure of glue lines

Adhesives are used for face-bonds between layers, bond lines between components and finger joints within the components of engineered wood products. Thermal stability of the adhesives influences the fire performance of engineered wood products compared to solid wood products. The fire behaviour is influenced by the chemical composition of each adhesive (Clauss, 2011) as well as the rate of heating, applied loading, and transient moisture conditions (Wiesner et al., 2021b).

Previous research found that even within each adhesive group (PUR, MUF, MF, PRF, and EPI adhesives) significant differences in performance could be observed (Klippel, 2014, Frangi et al., 2004). The assessment of adhesive performance at elevated temperatures has been done at both small (Kleinhenz et al., 2021b, Zelinka et al., 2019) and large scale (Kleinhenz et al., 2021a, Brandon and Dagenais, 2019) and with varying heating and loading conditions – which has led to conflicting outcomes and interpretations (Čolić, A. et al., 2021). A global consensus on the appropriate testing and understanding of bond line behaviour is lacking.

6. Research Gaps

Taller and larger timber buildings pose additional fire safety design challenges for engineers, architects and fire and rescue personnel. These buildings often utilise larger open spaces and larger timber members are required to carry higher loads that arise from the increased size and ambition.

Previous timber research has often been focused on fire resistance testing and it is not fully understood how outcomes from simply supported conditions of single elements in a furnace can be extrapolated to longer multi span systems, point support situations and more complex connection systems with higher performance requirements than existing test results. The question of connection and load details is especially pertinent when considered for the combined action of earthquakes and fires – two statistically dependent events.

The additional complexity in capacity also extends to the individual components of the structural timber elements – namely, how their composite action can be maintained. This requires better understanding of how adhesive bond lines perform the aforementioned complex loads but also under different fire conditions.

WG3.SG2.04

Property protection

Carl Pettersson, RED Fire Engineers (Sweden); **Annette Riel**, CREE GmbH (Austria); **Joachim Schmid**, ETH Zurich (Switzerland); **Daniel Brandon**, RISE, (Sweden); **Chamith Karannagodage**, ETH Zurich (Switzerland)

1. Introduction

With the increase in size and height of timber buildings, property protection in the event of fire becomes more important. The resulting damage after a fire can be extensive if appropriate fire protection strategies are not in place. Damage can be fire-induced but also due to water used in fire-fighting operations or burnt-off contents and fittings. Property protection and post-repair after a fire also relate to the insurance prospects of a building, to which the insurability demands an analysis of how the building might perform against fire and material and function recovery RISC Authority (2022).

2. Fire protection strategies and robustness

There are different types of fire protection measures which can be categorised into passive or active fire protection, which in combination with a particular building are part of its fire safety strategy (Buchanan & Östman 2022). Encapsulation of timber structures with non-combustible protective boards is an example of passive fire protection preventing the timber structure from being involved in a fire event. Provided with sufficient encapsulation, the fire can be expected to behave more similarly to a fire in a non-combustible building structure. Active fire protection measures such as sprinkler protection rely on different components to operate sufficiently or operate at all and therefore need particular attention in order to ensure that their installation, commissioning as well as management and maintenance are carried out accordingly.

Several experiments have proven that an automatic sprinkler system, as well as a water mist system, are effective in compartments with exposed timber in walls and ceilings (Frangi and Fontana 2005; Zelinka 2018; Kotsovinos et al 2022; Ko and Nour (unpublished)). Garis and Clare (2014) indicate, based on a statistical study, that operating sprinkler systems reduce the extent of fire spread in a building independent of its construction materials.

Nonetheless, fire is a systematic exposure, which affects a potentially large number of elements simultaneously Schmid et al. (2021). This makes fire behaviours and scenarios very hard to accurately predict. It is therefore of great importance that the fire safety in buildings has extensive robustness in all aspects of fire safety measures. The failure of one measure or component should not lead to the complete failure of the fire safety strategy in the building, and protective layers are necessary to prevent extensive damage, see Figure 1.

The scenarios for robustness analysis shall challenge the trial fire safety design by disabling active and passive fire safety systems, or common resources one at a time INSTA TS 950 (2014). Structural robustness also needs to be considered and general approaches for evaluating the structural robustness can be found in Schmid et al. (2021).

3. Firefighting strategies

A modern timber building with a combustible structure introduces complexity and a set of different possible fire behaviours which create practical challenges for the fire service. If combustible structures in a building are involved in a fire, longer burning and fire spread through openings (windows) can be expected. Fire spread into cavities in walls, shafts, floors and ceilings with combustible materials (see Figure 2, left) may also impose problems for the fire service. The outcome of firefighting operations in complex buildings will therefore depend on the fire service experience, training, routines, capabilities and techniques to mitigate the damage (Torero 2021).

Limited research has been conducted to find the most effective approach to locate and extinguish fires inside combustible cavities (Hox en Saeter Boe 2017) and timber structures (Brandon *et al.* 2021).

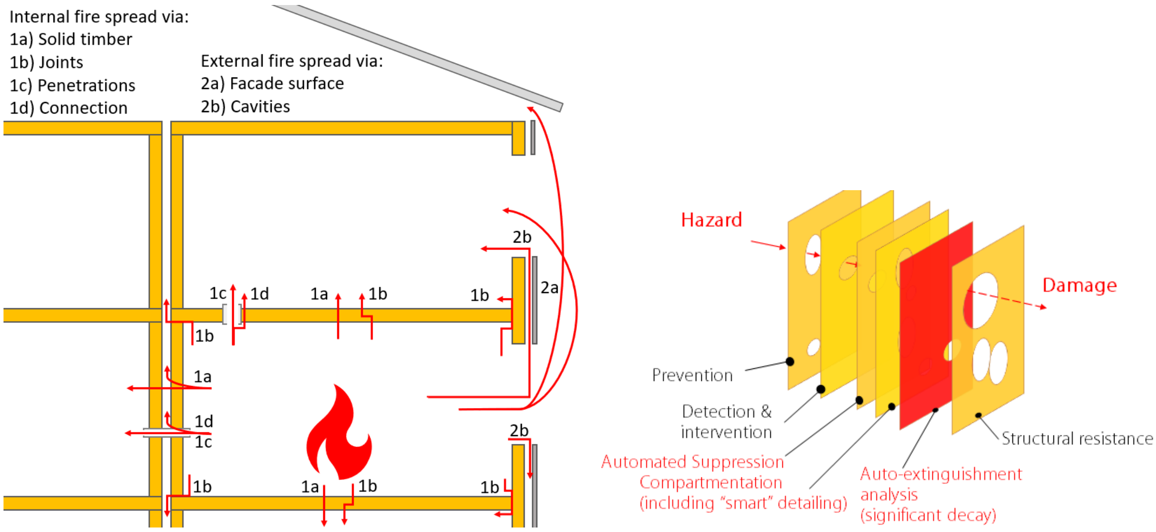

Figure 2: Left, potential paths of fire and smoke spread to be considered (information combined from Brandon et al. 2018); Right, visualization of fire safety elements to prevent from damage, modified from Schmid (2022).

4. Damage

Property damage resulting from fires generally results from the flame spread, smoke spread and extinguishment water and extinguishment activities. The influence of timber as a structural material on flame spread has been investigated in comparative studies with non-combustible building materials, by UK Dept of Communities and Local Government (2012), Garis and Clare (2014), Eriksson *et al.* (2016), Berg (2021), Brandon et al (2021b). The data indicated no statistically significant difference between the extent of flame spread in multistorey buildings in the US, Canada, New Zealand or Sweden. However, the further analysis did identify specific fire incidents where the presence of a timber structure was identified as a cause of excessive fire spread. Data from New Zealand indicated no statistical difference in the extent of water damage in terms of building areas affected (Brandon *et al.* 2018). However, data on the actual financial costs of these damages is missing.

Analysis of incident reports of a US database of high damage fires indicated that incidents with high levels of water damage were more frequently extinguished by the fire service than by sprinklers. Tests by Ko en Nour, (unpublished) analysed the water damage to exposed wood after sprinkler activation and concluded that connections between the floor and walls are sensitive to water damage. No published data on the influence of structural timber on smoke damage was found.

5. Post-fire repair

The repair of damaged components after a fire may be grouped into three main categories:

1. Non-structural repair, e.g. aesthetics or surface related repair;

2. Structural repair of parts of the components;
3. Structural repair by the exchange of components.

Typically, all categories are executed on site, which may require significant preparation (e.g. façade openings, bracing of the structure) of the work execution. Furthermore, the repair execution typically comprises the involvement of several professions (SCIUS Advisory 2020).

6. Property valuation

Publications from the scientific community in recent years have focused to a greater extent on mass timber. In comparison, US insurance companies address mass timber and lightweight construction in their publications (Hester 2022).

In a German publication from 2008, modifications of the underwriting evaluation criteria were still discussed with a focus on building class and fire resistance without mentioning or including water damage as a hazard (Stein 2008). Today, most publications on property protection also refer to the risk of water damage.

A recent white paper found that insurers lack knowledge, resulting in a lack of confidence in assessing the likely financial impact when a predominantly combustible building needs to be protected against fire (RISC Authority 2022). Furthermore, a trend toward "evacuation before collapse" and the blurring of the line between a collapse due to accidental damage and a collapse due to slower events as critical factors (RISC Authority 2022). It is important to know the golden rule of the provision of insurance for the people who deal with fire design and who do not, that is 'you can't insure what you can't quantify' (RISC Authority 2022). Insurers are rating the statistical likelihood of damage events taking place. Finally, the authors acknowledge long-lasting, trusting customer relationships that create the basis for reducing risk through collaboration using a mix of conventional and newer methods (RISC Authority 2022).

Our unmistakable recommendation to designers is to map the customer's design specifications to the relevant features (RISC Authority 2022) and track the insurance information requirements as presented in the reference.

Whether it makes economic sense to further reduce the extent of projected property damage could be analyzed by developing various concepts to support the client's profitability calculation. However, statistical analysis of firefighting and damage due to spread beyond the area of origin varies widely (Berg 2020, Brandon et al. 2018). One challenge is access to a complete data pool.

Underlining the urgency, a US/Canadian survey, that 20 people answered and 2 of them were project insurers, identified four information needs of the key influencer group “insurance and bonding”, one of which is the need of more research on durability and historical damage (SCIUS Advisory 2020).

The report (SCIUS Advisory 2020) is missing “property loss” as a type of insurance commonly used in construction projects, which could be interpreted as the trust in building regulations by insurance in the US/Canada being very high. Suggesting that there is a need to investigate why and what can be done to increase this confidence in other countries as well.

7. Fire safety during construction

There is a difference in statistical studies on how to limit fire damage during construction. Most research and guidance are based on light timber frame construction. One example being that timber buildings are prone to much larger fires during construction (UK Dept of Communities and Local Government 2012).

Several guidelines have been developed for fire safety during construction Bregulla *et al.* (2010), the UK Timber Frame Association (UKTFA, 2008) and Just *et al.* (2016), mostly based on experience, reasoning and good practice.

In July 2021, the French Prefecture of Police and the Paris Fire Department issued the so-called "Doctrine Bois", which presented new rules for multi-storey wooden buildings higher than 8 m, however, these are not yet incorporated into any regulation. These rules require the provision of fire extinguishers on each floor, 180 m^3/h hydrants, a dry standpipe in buildings over 18 m, an automatic fire alarm system and an automatic water sprinkler (*DTPP* 2021).

Since 2017, the Structural Timber Association (STA) in the United Kingdom has recommended compliance with the "16 Steps to Fire Safety" on construction sites where timber is used as load-bearing elements, continuing to provide guidance to insurers in 2021 (STA 2021).

In 2022, a new international guide emphasizes the necessary quality of craftsmanship, trade coordination, assignment of responsibilities, and key immediate actions in case of fire. Uninstalled or inactive fire protection equipment during construction makes it necessary to explicitly coordinate construction-related emergency measures with the assembly sequences (Buchanan & Östman 2022).

WG3.SG2.05

Comments and Questions COST Action Helen Meeting Gothenburg

This chapter lists the questions asked and comments given during the COST Action Helen meeting in Gothenburgh on the 4th and 5th of October 2022.

3. Questions

What design measures are the result of large-scale exposed timber surface fire tests pointing to?

<u>Answer during event: Effective measures are dependent on the compartment and the expected fire scenario. These can for example include protection of the walls, as recent studies show they can have more significant involvement in fires than ceilings. Furthermore, prevention of delamination of mass timber and failure of fire protective boards can be important.</u>

Which research answers sprinkler failure? Did you include references?

<u>Answer during event: A number of studies looked at the effect of sprinklers in specifically timber buildings. The most extensive work is performed by Professor Garris and studies Canadian fire statistics.</u>

Does exposing the timber elements have any plus sides aside from aesthethic/architectural added value? If the consequence can be so severe would it not just be logical to ever expose them?

<u>Answer during event: gypsum protection adds significantly to work load and costs. It also results in an increased carbon footprint. As timber is hygrospic it has a positive effect on room climate due to moisture buffering. Comfort and health benefits of having visible wood are often mentioned.</u>

Are there studies that indicate the extent of fire spread is more an issue for tall or large buidings? I guess we should be much more careful with fire spread and related issues in tall buildings (along facades or internal gaps etc.)

<u>Not discussed during event.</u>

Does the fire group also have an approach to a cross-over holistic consideration e.g. fire & repair, adaptability, connections, durability?

<u>Not discussed during event.</u>

4. Comment

A member indicated a clash between people protection and fire protection regarding fire safety.

Part 3

Blast Loads

Sub Group (SG) 3

WG3.SG3.01

High-strain rate effects in timber members

Abla Krouma, University of Ottawa (Canada); **Christian Viau**, Carleton University (Canada); **Ghasan Doudak**, University of Ottawa (Canada)

1. Introduction

Mass timber constructions are on the rise due to sustainability concerns, rapid construction, and cost efficiency. Since 2009, the number of tall timber buildings, defined by the Council on Tall Buildings and Urban Habitat (CTBUH) as having at least eight stories, has increased to sixty-six buildings (Council on Tall Buildings and Urban Habitat (CTBUH), 2022). Along with the increase in market share of the construction industry, this brings forth an increase in potential exposure to accidental and intentional blast explosions, such as that from gas leaks and vehicle bombs, respectively. Understanding how wood behaves when under high strain-rates and having well-established mitigation strategies to minimize the risk of progressive collapse, is required to ensure that these novel structures remain safe to building occupants. During the last two decades, the behaviour of various wooden structural systems under blast loading has been studied through numerous experimental studies. While some studies were conducted using live explosive testing, such as (Weaver et al., 2018), shock tube testing, which allows for the simulation of the shock wave of a far-field blast explosion (i.e. planar shock front), has allowed elements, connections, and sub-assemblies to be investigated whilst obtaining relatively amounts of quantitative results, such as reactions, deformations, and deflected shapes (Côté & Doudak, 2019; Lacroix & Doudak, 2018d; Viau & Doudak, 2021). Other studies have generated high strain-rates using split Hopkinson pressure bar (SHPB) (Bragov & Lomunov, 1997) or conducted impact tests using drop-weight impact apparatus (Sukontasukkul et al., 2000).

2. Wood material behaviour at high loading rates

Early studies on small clear-wood specimens under HSR (i.e., greater than 0.1 s^{-1}) observed apparent increases in wood's material properties (e.g., Liska, 1950; Gerhards, 1977), confirming the viscoelastic nature of the material. As an organic material, HSR effects and failure modes were found to be affected by the direction of the load and the moisture content (Widehammar, 2004). Under impact bending tests, the high shock resistant specimen developed long and coarse splinters specifically on the compressive side, while the normal wood specimen failed in shorter and fibrous splinters that are longer on the tension side (Kollmann & Côté, 1968), while brittle wood specimens failed in cross grain tensile fracture at lower impact levels.

3. Behaviour of full-scale timber elements

Some of the earliest experimental research carried out on the effect of blast loads on timber structures was conducted on model full-scale light-frame homes subjected to nuclear blasts (Kimbell & Fies, 1953; Randall, 1961). Post-test observations revealed that the failure of these structures was heavily dependent on continuity in the systems and was most often initiated at the load-bearing studs, roof hangers, and boundary connections. During the last two decades, the behaviour of various wooden structural systems under blast loading has been studied through numerous experimental studies. While some studies were conducted using live explosive testing (Marchand, 2002; Oswald, 2005; Weaver et al., 2018), shock tube testing, which allows for the simulation of shock waves emanating from far-field blast explosion (i.e., planar shock front) without the need for high explosives, has allowed structural elements, connections, and sub-assemblies to be investigated whilst obtaining relatively high amounts of quantitative results, such as reactions, deformations, and deflected shapes. This includes studies on light-frame construction (e.g., Lacroix and Doudak, 2015; Viau and Doudak, 2016a; 2016b; 2017), glued-laminated (glulam) timber elements (e.g., Lacroix and Doudak, 2018a), and cross-laminated timber (CLT) panels (e.g., Poulin et al., 2018).

4. Code implications for blast design

Both Canada and the United States have in recent decades developed initial editions of their respective blast design standards. The Canadian Blast Standard, CSA S850 (2012), provides guidelines and information with required performance criteria for buildings to resist blast loads, in addition to explosion types, blast wave, materials strength under blast loads, and guidance on design for steel, concrete, masonry, and wood components. There are four response limits for wooden element based on the type and are mainly based on elements ductility. In all the aforementioned shock tube studies, HSR effects were quantified through dynamic increase factors (DIF), which may be used by designers when designing these elements to resist blast explosions. Based on a review of the available experimental test data in tandem, the current versions of the American and Canadian blast codes assign a DIF value of 1.4 to all wood element types. While this value has been corroborated for solid sawn lumber products by several research studies, it has been found not to be applicable to other types of engineered wood products, such as glulam and CLT (Doudak et al., 2022).

WG3.SG3.02

Progressive and disproportionate collapse of wooden structures under blast effects

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1. Introduction

Starossek (2007) defines progressive collapse (PC) as one characterised by the disproportion between the triggering event and the resulting damage. Because of this, progressive and disproportionate collapse (DC) are often used interchangeably, however other sources distinguish these two phenomena (ARUP, 2011). PC happens when a chain of events occurs after the initial damage is directly caused by one another, and therefore the term characterises the type of collapse behaviour. DC on the other hand characterises the proportionality of the result to the initial damage, which can be arbitrarily defined. Therefore, a collapse event can in fact be either, both, and neither, however they often occur concurrently. The progressive and disproportionate collapse prevention includes non-structural protective measures (minimising probability of exposure to triggering event), reducing the risk of initial damage after the triggering event (key element design, overdesign), and designing for structural robustness. Robustness is defined by Starossek and Haberland (2012) as the ability of the structure to withstand the abnormal events without them resulting in disproportionate response. The topic of robustness in timber is investigated in more detail in the WG1. In progressive collapse research, usually an event-independent approach is taken, i.e., the specific exposure causes an initial damage is disregarded (Huber et al., 2018). In contrast, investigating the potential for progressive collapse after exposure to a blast explosion, requires an event-dependent approach.

2. Current standards and regulations

Progressive and disproportionate collapse prevention is directly addressed in Eurocode 1 Part 1-7 (CEN, 2006). The design approach there is divided between two groups of strategies:

1. Identifying the accidental actions and minimising the risk of damage through overdesigning the entire structure or reducing the risk of and vulnerability to the triggering event.
2. Assuming event independent localised damage and introduces alternative load paths, key element design and prescriptive rules for robustness such as ductility and integrity, all of which is encapsulated in Annex A.

It's worth noting that with respect to the second category, the only design calculation methods offered are the horizontal and vertical tie force calculations (assumed to provide sufficient redundancy for formation of alternative load paths), as well as key element design. This methodology considers various possible, albeit rare load scenarios, to which critical load-bearing elements may be exposed (Ellingwood, et al., 2007). Key element design is typically the method of last resort, utilized when alternative load path design does not generate a safe overall design of the structure (Cormie, et al., 2009). These elements are expected to be designed using a static-equivalent pressure of 34 kPa applied on either side of the element; a prescriptive requirement that has been derived based on the expected peak loads generated during the event of Ronan Point (ISE, 2010). This approach has been criticized for being simplistic and not appropriate for most design scenarios. No guidance on element design and dynamic analysis is provided, particularly in terms of high strain-rate effects, charge determination, and modelling methodologies. The code is written as material independent, approaching all buildings. With respect to large timber buildings, there are several issues with applying the current design guidance. Since the modern design codes are written based on reliability theory (Starossek, 2018), these actions and resistances have been based on statistically determined empirical data. This is clearly a problem when pushing the so-far-known limits of a material, as there is no reason to believe that the models developed previously will be appropriate.

3. State-of-the-art

Sørensen (2011) found that particular attention should be given to shear strength due to the brittle behaviour of timber in order to minimize the risk associated with progressive collapse in timber structures. Another consideration was ensuring that connections and main elements had sufficient ductility and strength in addition to ties consideration in the main principal directions. Furthermore, primary and secondary elements' ability to withstand reversal in load direction was identified as a key requirement for structural robustness.

The overwhelming majority of the reviewed literature has concluded that the effectiveness of prescriptive design methods is mostly unknown and overdesigning key elements should be avoided in lieu of performance-based approaches (Mpidi Bita et al., 2022). 127 structural collapse cases of Scandinavian timber buildings were investigated and 84% of the analysed structures were large-span timber structures, with instability being the most common failure mechanism (Frühwald et al.,2007; 2011). In the case of large span structures, increasing the redundancy and ties between elements could be detrimental to the stability of the system and can therefore be the direct cause of the progressive collapse (Starossek, 2006). Additionally, ensuring that connections and main elements have sufficient ductility and strength in addition to ties consideration in the main principal directions. While Alternative Load Path Analyses (ALPAs) have been conducted numerically, e.g. for a bay of a 6-storey CLT building in Huber (2021), a nine-storey mass timber building in Mpidi Bita and Tannert (2019), and a CLT floor system in Huber (2021), no literature on scenario-dependent ALPAs for blast exposures on multi-storey timber buildings is currently available.

A numerical study of a glulam frame was conducted using a non-linear dynamic analysis in the case of middle and edge column removal, however, the nonlinearity that came from materials and imperfections were not considered. The numerical analysis was conducted in three steps. The dynamic amplification factor value was between 1.9 to 2, which could be used in design verifications in linear elastic analysis case. Damping ratio and connection stiffness affected the value, while member stiffness had negligible impact (Cao et al., 2021).

WG3.SG3.03

Behaviour of timber connections under blast loading

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1. Introduction

Connections in timber structures, in contrast to steel and reinforced concrete, often have less strength and are generally more flexible than the members to which they connect. As a result, they are the focus of a vast body of research involving dynamic loading, such as seismic and blast. As timber is used for larger and taller public buildings, this area of research will become increasingly important to ensure public safety by limiting the effects of blast explosions. A wide variety of connection types in timber structures currently exists, including dowel-type connections include nails, screws, bolts and dowels, which are arguably most common way to create connections between timber elements on a construction site. Many different types of connections rely on dowel action, where a connector (usually steel) acts across a shear plane to hold two parts together. Examples include slotted-in steel plates with bolts or dowels passing through the timber and the plate (Dorn, de Borst and Eberhardsteiner, 2013; Reynolds et al., 2022), timber-to-timber connections where the connector passes through overlapping timber elements and steel hold-downs or angle-brackets which are secured to the timber with nails or screws (Ringhofer, Brandner and Blaß, 2018). Dowel-type connectors are also used to connect pieces of solid wood to create larger building elements, such as in nail-laminated (Hasan et al., 2019) or dowel-laminated timber (Bouhala et al., 2020; Sotayo et al., 2020). Specialist connection types seek to achieve particular elements of building performance. Connectors for seismic resistance take advantage of friction or yielding of a ductile “fuse” to dissipate energy in a connection and reduce the seismic load on the structure. As connections within between timber elements involve material behaviour in various orthogonal directions, strain-rate effects are expected to occur within these localized regions, however, their extent and magnitude has not been well documented, particularly for blast load.

2. Behaviour of timber connections under high strain rates

Various material properties of timber are strain-rate dependent, and as a result, connections exhibit strain-rate variations of strength and stiffness. “Medium” strain rates, as defined by Cheng et al. (2022), are possible in conventional hydraulic materials testing machines, but high strain rate loading representative of blast require a specialist setup such as a shock tube (McGrath and Doudak, 2021). Disproportionate collapse scenarios may see HSR effects, but there is no clear consensus on the appropriate loading rate for these experimental test programs (Mpidi Bita and Tannert, 2020). Early research on nailed timber joints involving medium strain rates showed strengths approximately 30% higher at a high rate of loading than quasi-static (Girhammar and Andersson, 1988). Various increase in strength and stiffness were found, however, it was noted by the authors that these increases were greatly affected by the slenderness ratio of the nailed joint. In a similar study, monotonic loading of bolted timber joints at two different loading rates, a slow rate (0.042 mm/sec) and a fast rate (25 mm/sec), in both parallel- and perpendicular-to-grain directions, were conducted (Daneff, 1997). The joints in question consisted of two-member wood single-bolt joints. The author concluded that connections exposed to a fast rate of loading exhibit stiffer behaviour and attained higher yield loads than those exposed to a slow rate of loading (Daneff, 1997), however, it was noted that these increases were heavily affected by the slenderness ratio of the connection as well as the direction of loading (parallel versus. perpendicular to grain).

3. Design of connections against blast loading

Sufficient connection ductility and strength under high strain rate loading is required for a timber system to resist blast loads. As in the case of connections for seismic resistance, the strategy when it comes to connection of timber assemblies under blast load is to concentrate the irreversible deformation in ductile connectors, such that damage to the load-bearing timber element is mitigated. Recent work has focused on common types of connections for light-frame wood stud walls. Viau and Doudak (2016a) investigated typical prescriptive code guidelines for light-frame wood stud walls and concluded that the typical nailed connections, including those designed for high seismic regions, did not allow the studs to develop their full flexural capacity due to a premature failure, and thus resulting in hazardous debris. While significant damage in the joist hanger connections was observed, the studs were able to attain their ultimate flexural resistance. It was concluded that overdesigning the connections based on the stud capacity may not be adequate and that proper understanding of the failure mechanism of the connection must be investigated. In regard to mass- and heavy-timber, bolted connections in glulam assemblies (McGrath and Doudak, 2021; Viau and Doudak, 2021b), as well as self-tapping screws and angle bracket connections used in CLT construction (Côté and Doudak, 2019; Viau and Doudak, 2019) were investigate through small- and full-scale testing were conducted through the use of a shock tube.

While evidence of HSR effects has been found in wood connections, further research is required in order to further refine blast design codes in regard to wood connections. Properly detailed connections as well as those designed to fail via ductile mechanism (i.e., wood crushing, yielding of steel) allowed the timber assemblies to withstand larger blast loads when compared to overdesigned connections (Côté and Doudak, 2019; Viau and Doudak, 2019; 2021b). As in the case of connections for seismic resistance, the strategy is to concentrate the irreversible deformation in ductile connectors, so that the relatively brittle timber material is left undamaged. This concept has been applied in isolation (Wang et al., 2017), within precast concrete assemblies subjected to blast loads (Lavarnway and Pollino, 2015), and within glulam and CLT assemblies subjected to blast loads (Viau and Doudak, 2021c). In the latter, energy-absorbing connections (EAC) were designed to ensure that yielding in the EAC always occurred prior to failure of the wood element through capacity-based design. From experimental testing, the implementation of these connections allowed for upwards to twice the amount of blast impulse imparted prior to damage occurring in the wood element, when compared to overdesigned end connections.

WG3.SG3.04

Modelling of blast-loaded timber elements and assemblies

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1. Introduction

The violent transient nature of blast scenarios poses a challenge for building accurate and reliable numerical models of the effects on structures after explosions. The literature concerning modelling of this phenomenon is mainly treating reinforced concrete and structural steel systems. Given the relatively recent trend of building multi-storey timber buildings, investigations regarding wood are scarce when compared to other materials. This report attempts to review of the state of the art concerning modelling considerations of timber structures with regard to blast exposures. In general, numerical models for blast include models based on the Finite Element Method (FEM) and simplifications such as Single Degree of Freedom (SDOF) and Two Degree of Freedom (TDOF) models. As live explosion testing tends to be logistically difficult and expensive, being able to generate and validate numerical models tends to be the preferred approach to study these relatively novel structures under high magnitude, short duration blast loads.

2. Finite element modelling of blast scenarios

The main challenge in conducting finite element modelling regarding effects of a detonation on a structure is that two coupled phenomena need to be considered; i) the explosion's rapid compression of its surrounding fluid (air) with its subsequent pressure shock front, and ii) the structure's response to the shock front, which in return affects the propagation of the pressure wave. Numerical models may often investigate these phenomena in two separate types of models; the blast loads are predicted by models based on Computational Fluid Mechanics (CFD), and the effects on the structure given the loads are predicted by models based on Computational Solid Mechanics (CSM) (Ngo et al., 2007). These uncoupled models usually overestimate the loads, since the structure is assumed to be rigid in the CFD model, whereas in *coupled models,* both phenomena are accounted for simultaneously. The latter tends to provide more accurate results, but is computationally more expensive. In lieu of CFD, the approximate pressure-time history for a given charge weight and standoff distance on a component can be modelled by a triangular-shaped (see Figure 1). These curves can be obtained from what is referred to as the Kingery-Bulmash Blast Parameters, which were empirically derived from explosion testing in free-air (Kingery and Bulmash, 1984).

Finite element models for blast analysis can be made in several commercial software packages, examples include AUTODYN, DYNA3D, LS-DYNA, ABAQUS (Ngo et al., 2007). LS-Dyna, in particular, has extensive documentation on a Wood material model, including strain rate effects, hardening, softening (damage modelling), temperature, moisture content, and user-defined material parameters. ABAQUS does not include specific constitutive laws for wood materials by default, however, this modelling environment can facilitate the analysis of damaged configurations based on inclusion of orthotropy, strain rate effects, hardening, and damage laws that should be properly calibrated.

In numerical models, including simplified models (SDOF and TDOF) and FEM, time needs to be discretised into finite timesteps. For transient problems like blasts, a response history analysis is performed, i.e., the equations of motion governing a system are solved by *direct integration* along these timesteps (Cook et al., 2002). Direct integration can be performed by *explicit or implicit* algorithms. For explicit integration, the state of the system at a subsequent time step is simply extrapolated using only information from the state of the system at its previous time step, which makes each step computationally cheap. For implicit integration, the state of the system at the subsequent time step is calculated accounting for both the state of the previous *and* the subsequent step, which usually involves a Newton-Raphson solution scheme including the computationally expensive conversion of the stiffness matrix. Refer to Huber (2021) for more explanation.

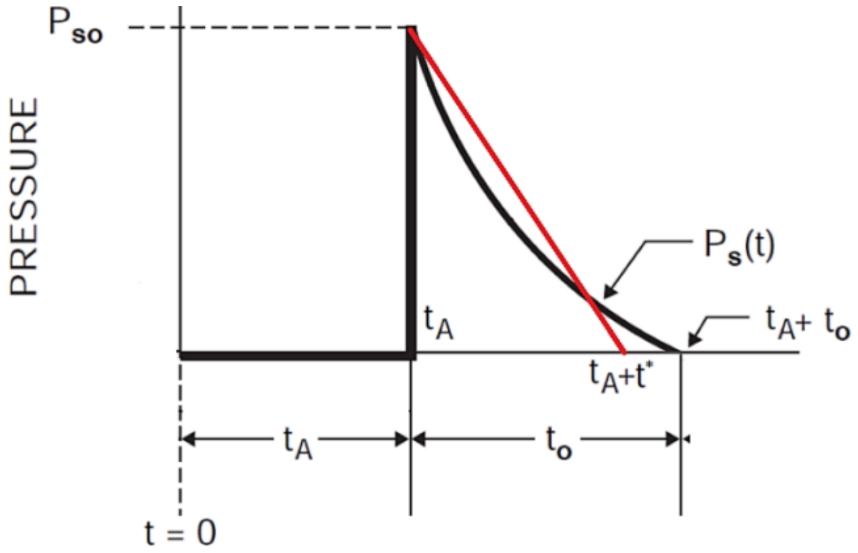

Figure 1: Triangular-shaped curve to model a blast load (adapted from Dusenberry, 2010)

3. Simplified modelling of timber structures subjected to blast

Due to the many uncertainties associated with materials and blast wave characteristics, most of the modelling conducted on wood structures subjected to blast loads has been limited to simplified modelling such as Single-Degree-of-Freedom (SDOF) and Two-Degree-of-Freedom (TDOF) modelling. These approaches consist of idealizing the actual structural element and/or assembly into lumped-mass systems. The use of the former has been proving capable at modelling wood studs (Jacques et al., 2014), stud walls (Lacroix and Doudak, 2015), glulam beams and columns (Lacroix and Doudak, 2018a; 2018b), and CLT panels (Poulin et al., 2018) under blast loads, whereas the latter has been used and validated against experimental full-scale test data for light-frame stud wall with nailed connections (Viau and Doudak, 2016), glulam elements with bolted connections (Viau and Doudak, 2021b), CLT assemblies with self-tapping screws and bearing angles (Côté and Doudak, 2019; Viau and Doudak, 2019), and energy dissipating connections (Viau and Doudak, 2021c) under simulated blast loading (Viau and Doudak 2021a).

WG3.SG3.05

Comments and Questions COST Action Helen Meeting Gothenburg

This chapter lists the questions asked and comments given during the COST Action Helen meeting in Gothenburgh on the 4^{th} and 5^{th} of October 2022.

1. Questions

There isn't any specific Code for blast design on timber buildings in Europe, is it?

What design methodology would (in general) prefer for designing TTBs against blast - robustness or dedicated connections?

Can the connections for blast robustness maybe also be used and beneficial with respect to low damage seismicity, adaptability, repairability aspects etc?

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