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The structural capacity of laminated timber compression elements in fire: A meta-analysis

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ABSTRACT

Modern building construction is increasingly applying laminated timber products as structural members for larger and more ambitious projects, both commercial and residential. As a consequence, designers require reliable knowledge and design tools to assess the structural capacity of laminated mass timber elements in fire. This paper reviews and assesses available data and methods to design for fire resistance of laminated mass timber compression elements. Historical data from fire resistance tests is presented and compared against the available design calculation methods. The underlying assumptions of the thermal and structural analyses applied within the presented calculation methodologies are discussed. The resulting meta-analysis suggests that the available methods are all able to make reasonable predictions (with an average mean absolute error (MAPE) of 22% across methods) of the fire resistance of glued-laminated columns exposed to standard fires; however, the available methods for CLT walls give inconsistent (MAPE of 46% across all methods and 30% excluding extreme outliers) and potentially non-conservative results (up to 88% of investigated cases are statistically non-conservative). Additional research on loaded compression elements is therefore needed.

1. Introduction

Driven by sustainability and aesthetic factors, there has been a steady increase in the use of timber for multi-storey residential and commercial buildings in recent years. The use of engineered laminated timber products has allowed the design and construction of increasingly larger and taller buildings that use timber as the primary structural material [1–7]. Timber is a renewable material with a favourable strength to weight ratio [8], and the production of laminated timber products requires less energy input than concrete or steel [6,9]. The carbon that is sequestered during timber growth is assumed to be locked into the resulting products [10]. Timber buildings are also lighter than equivalent steel or concrete structures, and lend themselves to offsite modular construction, which presents significant advantages for urban residential construction projects.

As timber building heights increase, the structural demands on vertical compression members become more critical due to both gravity and wind loading effects [6]. This increases the relative cost of the structure, but also offers significant opportunities to sequester carbon into structures by using timber, and therefore allows more sustainable building designs. However, because timber is a combustible material – unlike

masonry, concrete, or steel – fire resistance concerns are commonly cited as an obstacle, discouraging the use of engineered mass timber for multi-storey buildings [11–13]. Indeed, in many jurisdictions the use of exposed structural timber is limited by building height or total number of storeys, largely as a consequence of the historical development of prescriptive building codes based on minimising or limiting the use of combustible structural materials [14]. Despite this perception and regulatory limitations, mass timber generally has a good reputation as regards its fire resistance; this is due to the insulation provided by a sacrificial char layer that forms during fire exposure, which protects the underlying timber from further heating and pyrolysis, and its predictable charring rate when exposed to standardised temperature time curves [15, 16].

Because timber is combustible, it is important that designers of tall and medium rise buildings be confident that the conditions exist for timber to achieve self-extinction once the fire load in a compartment has burnt out [17,18]; therefore fire compartmentation must be maintained throughout the full duration of a fire. For this and other reasons, load bearing timber members must not fail for the full duration of a building fire, i.e. after burnout of the combustible contents and until the structure has cooled.

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A review of the design methods for the structural fire performance of timber elements has been given previously by White [19]. The current paper extends this work and presents a state-of-the-art review of the performance of laminated mass timber compression elements exposed to furnace testing, with a focus on assessing the available design and calculation methods in comparison with the results of furnace tests on such elements available within the literature. The aim is to provide an overview of the key issues that ought to be considered for fire-safe design of structural timber compression members, and to ensure that available knowledge and tools are suitable for the more ambitious engineered mass timber buildings of the future.

2. Previous test results

This section reviews available fire test data for timber compression elements. All of the reviewed tests are standard fire resistance tests, in which fire resistance durations are recorded for loaded members in fire testing furnaces, following a standard temperature time curve such as the ISO-834 curve [20]; or a similar/equivalent fire curve. Any reference in this paper to a standard fire or a standard time temperature curve refers to a temperature development as defined in ISO-834 [20], ASTM E119-16a [21] or EN 1363-1 [22]. While there are small differences in the temperature developments between these standardised fire curves, this is unlikely to have a significant effect due to the high Biot number of structural mass timber elements. The main difference between these standards is most likely the measurement (and therefore) control of the gas temperature within a test chamber (furnace). While ASTM E119 requires thermocouples in tubes, EN1363 requires the use of plate thermometers. This can vary the thermal insult on the tested elements, however, this is not the subject of the current paper. Any older national standards are thus considered broadly equivalent to the standards described above.

When considering the data from available tests on structural compression elements made from wood, it is important to differentiate between tests on solid mass timber versus those on laminated timber products. The former are cut directly from bulk timber and are therefore limited in size and quality by the properties of the tree from which they originate. Within the broad class of laminated mass timber products there is further differentiation between glued-laminated products, where successive timber laminations are typically orientated all in the same grain direction, and cross-laminated timber (CLT) products, in which timber lamellae are arranged in a cross-wise manner in alternating succession, and which are typically intended to be used for floor slabs and wall panels where loads in more than one direction need to be resisted.

2.1. Available data on solid timber columns

The fire resistance of timber, as assessed by standard temperature time tests in furnaces, has been studied since the early 20th Century. Ingberg et al. [23] describe a series of furnace tests on loaded solid timber columns as part of a wider test series on columns made from various construction materials. Their test results show that the timber columns, loaded to ‘approximately 10% above the working load’ (since these tests were undertaken in a time when design was dominated by the ‘working stress’ design philosophy), generally retained their load bearing function longer than unprotected steel columns when subjected a standard heating curve under similar sustained loading levels. It was observed in these early tests that the connection details between the columns and the test loading ram were critical, and that the timber columns failed near to steel (or cast iron) caps that were used to apply the loads and were placed inside the furnace test chamber. A similar observation was made by Neale [24], who, in a series of furnace tests on solid timber columns, noted that timber columns with steel bearings located inside the furnace chamber failed much earlier than comparable test specimens with concrete support bearings. Neale [24] also noted that most of the columns failed near

‘defects’ in the timber and that all columns experienced ‘a degree of plasticity’ before failure.

Stanke et al. [25] tested a number of solid timber columns, yet they reported that their results were ‘inconsistent’, and therefore not useful, apparently due to shrinkage cracks in the timber causing accelerated charring and resulting in lower fire resistance ratings than expected. Peter and Göckel [26] also reported the formation of shrinkage cracks leading to accelerated charring in stocky solid timber compression elements, yet they reported that the fire resistances of the tested columns were all essentially as expected, and within $\pm 10\%$ of one another. Considering the above issues, results from furnace tests on solid mass timber columns are not included in the meta-analysis presented in the current paper.

2.2. Available fire test data on engineered timber in compression

2.2.1. Glued-laminated columns

Laminated mass timber construction elements can be produced in larger sizes than solid timber, and from a purely fire resistance perspective can sometimes be used without fire protection (encapsulation); i.e. where it is assumed that the rate of charring of the cross-section is known, and that a residual cross section can be approximated which is able to maintain the load bearing function for the requisite duration of standard fire exposure under the fire limit state loading. An extensive study of the fire resistance of uniformly exposed (i.e. all four sides) glued-laminated columns is presented by Malhotra and Rogowski [27], who also developed empirical relationships to predict fire resistance times depending on the species, glue, shape, and level of sustained loading (this is discussed in more detail in Section 3.3). Stanke et al. [25] present standard fire tests on glued-laminated (and solid timber, as noted previously) columns to assess their fire resistance under various sustained loading levels. They found that the columns normally failed by global buckling and that, due to the instability-dominated failure mode, the shortest edge dimension was the most critical parameter impacting fire resistance for rectangular glued-laminated columns. Fackler [28] reports results from furnace tests on two glued-laminated columns, one manufactured using ‘adhesive based on melamine’ and one using a ‘urea formaldehyde based adhesive’. Both columns failed by global buckling after standard fire durations of about 48 min. Unfortunately Fackler [28] does not give more specific details on the glues used. Haksever [29] also describes tests on glued-laminated columns under standard fire exposure, and demonstrates that the dominant failure mode is global buckling with the rate of deflection before failure apparently being independent of the column dimensions and slenderness; good repeatability was noted for tests performed under identical conditions. Finally, Peter and Göckel [26] performed tests on stocky solid timber and glued-laminated columns that were dimensioned according to DIN 4102-22:2004-11 [30] to avoid buckling failure and ensure crushing. In this case, separate unloaded reference specimens were also placed within the same testing furnace during fire exposure, to assess the charring rate. The results from the sources described above will be used in the meta-analysis in Section 4.1.

2.2.2. CLT walls

For CLT compression elements exposed to fire, prior research has also concentrated on fire resistance tests of CLT panels in furnaces, and primarily on their associated charring rates when subjected to standard heating curves [31,32]; this includes studies of their structural response when either protected or unprotected.

Schmid et al. [33] performed seven standard fire tests on CLT wall elements under sustained load with fire exposure from one side and showed that, due to the potential for instability failures, their own models for the load bearing capacity of CLT walls were sensitive to small changes in the size of the residual cross section of only a few millimetres. Similarly, three wall panels tested under sustained loading in a subsequent standard fire testing programme on CLT walls by Osborne et al. [34] failed in global buckling, where the mid-panel deflection grew

steadily during heating and eventually led to instability due to accelerating secondary moments and reductions in the panels' flexural rigidity due to surface charring and increasing thermal penetration depth.

Six loaded furnace tests on CLT wall panels, with differing lamellae build-ups and boundary conditions, were performed by Klippel et al. [35]; however, these yielded no conclusive results as regards structural fire resistance since no obvious trends could be identified within the measured deflection responses during the tests, and all tests were unfortunately stopped prior to structural failure.

Suzuki et al. [36] present results from tests on eight loaded and eight unloaded CLT wall panels with different lamination build-ups, effective buckling lengths, load levels, and slenderness ratios. These tests also show that buckling is the dominant failure mode, since the effective slenderness and load eccentricity increase with progression of charring (i.e. advancement of the in-depth pyrolysis front). Deflections were observed to effectively plateau when progression of in-depth heating affected the weak, crosswise orientated layers in the CLT, and only increased when the parallel strong layers were subsequently affected on further charring. This effect was not observed in Suzuki et al.'s [36] work for load levels above 33% of the ambient temperature ultimate buckling load (predicted using Young's moduli from ambient reference tests), since for these load levels the walls' failed due to global buckling after pyrolysis of the first strong layer (i.e. the surface layer).

Individual test reports from manufacturers or timber associations have also been reviewed in drafting the current paper, and are assessed in conjunction with data from producers and from the Standard for Performance-Rated Cross-Laminated Timber (PGR 320) [37]. The Canadian Wood Council [38] commissioned a standard fire test on a Type X gypsum board protected three-layer CLT panel, which, with 200 kN/m axial load applied, achieved a standard fire resistance rating of 66 min before structural global buckling failure was reported. A 175 mm thick, five-layer CLT wall panel with Type X gypsum board protection was tested with 127 kN/m axial load applied by the American Wood Council and achieved a fire resistance rating of 186 min, after which the wall panel failed structurally [39]. The temperatures between the gypsum board and the CLT was 300 °C after 24 min of heating.

3. Review of predictive calculation methods

Multiple methodologies are available in the literature to predict the structural fire resistance of mass timber compression elements (i.e. columns and walls) when subjected to standard fire exposures in furnace tests. The available methods are reviewed and assessed in this section. Some of the methods are empirically based, whilst others are underpinned at least partly by engineering mechanics; some of these were explicitly developed as prescriptive design guidance, and are therefore inherently conservative, whilst others were developed to predict fire resistance rather than be conservative *per se*. The predictive performance of the respective models is compared in Section 4 against the available test data found in the literature.

3.1. Method requirements

All methods aiming to analytically predict the fire resistance of timber compression members need to employ several assumptions and sub models; these relate to (1) the effect of fire on the temperatures over the cross section and its remaining strength, and (2) an assessment of the load bearing capacity.

3.1.1. Charring

Pyrolysis of timber yields char, which has negligible mechanical strength but a low thermal conductivity and thus provides thermal insulation to the wood within the interior of a fire-exposed timber element. Charring and sacrificial loss of cross section form the basis of structural fire design for unprotected timber elements. A 300 °C isotherm is typically assumed to demarcate the boundary between charred and

uncharred timber [9,32,40,41]; slightly different temperatures of 280 °C [42], 288 °C [43,44] or 260 °C [36], are also quoted in the literature. A convenient property of charring rates for timber in standard temperature time exposures is that, after initial short-lived peaks in the charring rate, it settles down to an approximately steady-state. For structural fire design calculations it is widely assumed that the charring rate remains essentially constant, since an essentially steady state balance is reached between char loss due to oxidation and progression of the charring front during standard fire tests in furnaces [9,45]. However, the different fire resistance prediction methodologies reviewed in the current paper assume slightly different average charring rates as the basis of their calculations. When glued-laminated columns are exposed to three- or four-sided heating, corner rounding occurs due to non-uniform thermal gradients and increased charring near the corners. Various methods have been proposed to account for corner rounding, with some methodologies accounting for corner rounding implicitly, by assuming an increased notional charring rate and neglecting corner rounding in subsequent calculations (i.e. assuming square corners).

3.1.2. Heated uncharred timber

Char has negligible strength and hence should be ignored in structural calculations. Beneath the char layer, however, a zone of heated timber also exists with reduced mechanical properties; this zone is typically called the 'thermally affected zone' or 'heat affected zone'. A comprehensive review of the loss of (a) strength and (b) stiffness of heated timber is given by Gerhards [46]. The Gerhards review highlights the influence of the initial and transient moisture content on the reductions of strength and stiffness experienced. For example, Fig. 1 shows reduction curves with temperature for softwood timber compressive strength and stiffness (parallel to the grain) presented by various researchers. Clear disagreement is evident between the various curves; this likely stems from the fact that different researchers will have used various different experimental set-ups, and due to the implicit inclusion of species type, timber grading, moisture migration, and creep, amongst other factors [40]. It should be noted that the close agreement between the Young & Clancy and Jong & Clancy curves in Fig. 1 b) may be caused by the participation of the same author in both studies (as opposed to these curves having a higher acceptability in the fire safety community).

3.1.3. Instability

For timber compression elements exposed to fire, instability (i.e. global buckling) is likely to be a critical failure mode, particularly since the effective slenderness of timber compression elements can be expected to increase during heating. Since Euler's buckling formula assumes an imaginary, perfectly linear and homogenous column, in reality, given the existence of inherent eccentricities, buckling failure invariably occurs at loads below the Euler buckling load. Methods aiming to model the mechanics of slender compression elements need to account for instability failures, which are likely to occur before the actual crushing strength of timber is reached.

3.2. Available prediction methods used in meta-analysis

The following sections describe six methods that have been proposed in the literature to calculate the fire resistance of glued-laminated columns and/or CLT walls in compression. All of these have been included in the meta-analysis presented later in this paper. For completeness, Section 3.3 provides short descriptions of other methods to predict the fire resistance of timber compression elements in the literature, however these additional methods have not been included in the meta-analysis, since in all cases insufficient information is given by the authors to fully reproduce their results.

3.2.1. Lie's method

Lie [54] developed a semi-empirical method to predict the fire resistance of mass timber columns under standard heating exposures by

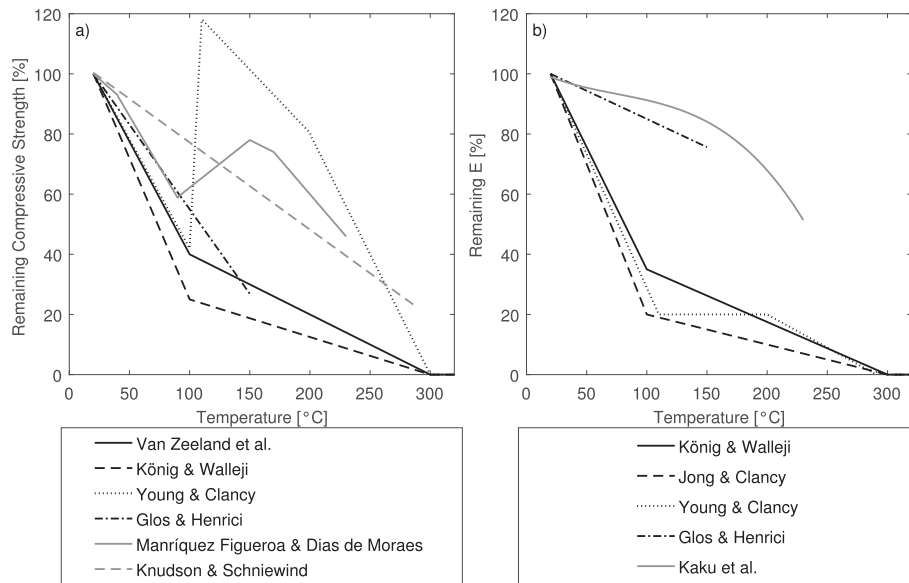


Fig. 1. Normalised reduction curves with temperature for: a) compressive strength of timber parallel to grain from various authors [41,47–51], and b) elastic modulus in compression for timber parallel to the grain, also from various authors [41,47,51–53].

calibrating a range of solutions to a simple mechanical model. Lie's model, underpinning the derivation of Lie's method [54], is given by Eq. (1):

$$\frac{k}{\alpha} \cdot \frac{B/D}{d/B - (1 - B/D)} = \left(\frac{d}{D}\right)^n \quad (1)$$

where k is the applied load as a fraction of the ultimate ambient capacity. α is a blanket strength reduction factor of 0.8 to the strength and modulus of elasticity of the remaining (uncharred) cross section, which also implicitly accounts for corner rounding. Since corner rounding is accounted for in α , a constant charring rate of 0.6 mm/min is assumed throughout the burning duration. Variables B and D are the initial larger and shorter side dimensions of the column, respectively, and d is the reduced (due to charring) dimension of the shorter side. Crushing and buckling failure modes are differentiated through application of an exponent, n , which is stated as being 1.0 for stocky columns, 3.0 for slender columns, and 2.0 for intermediate columns, although no guidance is given as to how a column should be classified into any of these three groups and in the end the method simply assumes that all columns can be treated as intermediate. With the above variables, a range of solutions were computed and used for fitting a simplified empirically-based equation to determine the fire resistance t_f (Eq. (2)), where an additional factor, f , is introduced that depends on the length-to-depth ratio of the column (this was apparently needed to calibrate the equations to better match the experimental data, combining n and k):

$$t_f = 100fD \left(3 - \frac{D}{B}\right) \quad (2)$$

Lie proposes six possible tabular values for f between 1 and 1.5; increasing with reductions in the applied load (in proportion to the permissible load) and/or a reduction in the column's length-to-depth ratio.

3.2.2. EC5 method

The procedures outlined in the harmonised guidance in EN 1995-1-2 (EC5) [55] are not strictly applicable to CLT elements in fire, yet it is likely that practicing engineers may apply them to these cases regardless. EC5 [55] advises designers to use the Reduced Cross Section Method (RCSM) to determine the remaining effective cross-section of wood based structural products (e.g. glulam or LVL) during standard fire exposures.

In this approach the reduced mechanical properties of the heated timber below the char layer (with a depth of about 30–40 mm under standard fire exposures [56,57]) are lumped into an additional assumed depth of zero strength timber beneath the char. This zero strength layer (ZSL) further reduces the effective size of the cross-section as fire exposure progresses. The remaining residual cross section is, for calculation purposes, assumed to be at ambient temperature, and the ambient temperature design guidance for timber structural elements from EN-1995-1-1 [58] is used to determine its capacity. A ZSL depth of 7 mm is currently suggested in EC5 [55] and was originally derived from studies on glued-laminated beams in bending [57]. Recent studies have cast doubt on whether this value/approach is suitable for different loading conditions [59,60], heating conditions [61], or for CLT more generally [59, 61]. In addition to the RCSM, EC5 also contains a reduced properties method, which reduces the strength and stiffness of heated timber based on a section factor. This method is due to be removed from future editions of EC5 and is not permitted in some jurisdictions. In addition, it is considered less accurate and cannot be applied to walls or slabs [40]. This method is therefore not considered in the current analysis.

EC5 [55] assumes an average one dimensional charring rate of 0.65 mm/min for solid or glued-laminated timber. If corner rounding is to be expected (i.e. for glued-laminated timber columns exposed from more than one side) the charring rate is increased to a notional charring rate of 0.7 mm/min. An extended charring model for CLT has been proposed by Klippel et al. [62] and this model is intended to be implemented in the upcoming revision of EC5. This model is therefore used herein to calculate charring rates for CLT walls. The charring rates are calculated from the multiplication of numerous k factors, which take into account the gap width, orientation, and expected falling off of charred layers [62,63]. It is assumed that all considered CLT panels in this study have a gap width of 2 mm or less. Therefore the charring rate for CLT walls constitutes 0.65 mm/min where only the outermost layer is charred and 0.8 mm/min where charring progresses past the first layer.

If protective gypsum board claddings are used, EC5 calculations can be added which assume that the applied protection delays charring until a calculated fall-off time, and the charring rate is increased thereafter. For gypsum board of types A and H the fall off time is assumed to be equal to the time to onset of charring of the timber behind it, which (for one layer of type A, F, or H gypsum board with thickness h_p) can be calculated using Eq. (3).

$$t_{ch} = 2.8h_p - 14 \quad (3)$$

To account for buckling in the structural capacity evaluation, EN-1995-1-1 [58] imposes a reduction factor, k_c , which is applied to the ambient temperature compressive strength of the timber. This factor is dependent on the element's slenderness and the relationship between its strength and its flexural rigidity. Additionally, the reduction factor is influenced by a variable, herein denoted c , to implicitly account for inherent (accidental) load eccentricities depending on the type of timber used. For glued-laminated timber c is 0.1 in EC5 and it is assumed that this value can also be applied to CLT (in the absence of specific guidance stating otherwise). This is derived from a constitutive relationship proposed by Ylinen [64] that accounts for deviation from Hooke's Law in the relationship between stress and strain in the inelastic range. To achieve this, the 'straightness factor', c , is taken as a function of the proportional limit, the elastic modulus, and the yield strength of the material.

3.2.3. The fire safety in timber buildings (FSTB) method

The Fire safety in timber buildings (FSTB) report [65] was produced as an unofficial technical guideline for Europe that provides amended guidance to EC5 for the determination of fire resistance for various timber products. The additional guidance relates to protective encapsulation, as well as weakening through a compensation layer (analogous to the zero strength layer in EC5) of CLT in standard fire exposures. For the progression of the char front FSTB [65] states that char ablation (i.e. r falling off) is less pronounced for vertical elements as compared with horizontal elements, and can therefore be ignored for walls or columns, although the experimental evidence for this design approach appears to be somewhat limited [65]. The ZSL (called a 'compensation' layer) for CLT in the FSTB report is assumed to depend on the element type (floor or wall), the loading condition (tension or compression side), and the overall thickness and number of lamellae. For example, for an unprotected five-layer wall, the depth of the ZSL is determined using Eq. (4):

$$ZSL = \frac{h}{15} + 10.5 \quad (4)$$

where h is the overall thickness of the slab. For a fixed value of h , the ZSL increases as the number of layers increases and with the addition of protective claddings (as the heating through protective claddings can occur before the onset of charring).

In addition to the guidance on protective claddings in EC5 [55], FSTB [65] alternatively offers the calculation of fall-off and start of charring times based on a database of gypsum board test results from across Europe. Since FSTB is considered as a companion document to EC5 [55], all other required calculation aspects (e.g. structural assessment) are assumed from EC5 [55,58] in the current study. As for the EC5 method the extended charring model by Klippel et al. [62] is considered for the FSTB method.

3.2.4. NDS method

The National Design Specifications (NDS) [43] for wood construction in the USA recommends a mechanics-based calculation model in which the remaining load bearing capacity in fire is assessed against the applied load for CLT and glulam amongst other wood products. The NDS method [43,44] assumes that the average charring rate is dependent on the burning duration, with a basic one-hour charring rate of 0.635 mm/min. This approach attempts to implicitly account for the charring rate peak experienced during the early stages of a fire, however it fails to account for the initial accelerated loss of cross section. This would only affect outcomes for smaller structural elements or fire resistance ratings of less than one hour, which are unlikely to apply for tall timber construction, but for smaller. Additionally, the NDS [43] provides empirically derived equations that can be used to adjust the charring depth, a_c , to account for the influence of falling off of charred of CLT lamellae during heating; this (Eq. (5)) depends on the number and thickness of lamellae as follows:

$$a_c = 1.2 \left[\sum_{i=1}^{n_l} h_{l,i} + \beta_n \left(t - \sum_{i=1}^{n_l} t_{gl,i} \right)^{0.813} \right] \quad (5)$$

where n_l is the number of lamellae, h_l is the depth of each lamella, and $t_{gl,i}$ is the time taken to char through a lamella to the glue-line. To account for the strength loss of heated timber, the NDS approach [43] artificially increases the charring rate 20% above the applicable nominal charring rate, to implicitly account for the loss of strength of heated timber; this 20% increase is also assumed to implicitly account for corner rounding [44].

Similarly to EC5 [58], the NDS method reduces the strength of the remaining cross section (assumed to be at ambient temperature since heating is accounted for implicitly through an increased charring rate) by multiplication by a factor, k_c , to account for the effects of instability. This factor depends on the slenderness, the ratio of strength to stiffness of the assessed wall or column and a straightness factor c , which is given as 0.9 for both glued-laminated timber and CLT. Due the manner in which the equations are formulated the influence of c of 0.9 on the buckling capacity in NDS is the same as for a value of c of 0.1 in EC5 [58], i.e. 10%, for both methods. If all partial safety and adjustment factors are stripped out of these equations, k_c , is equal for the NDS and EC5 methods, as shown in Fig. 2, which also shows that the reduction is more severe for higher strength-to-stiffness ratios. The NDS [43] only provides a method for predicting the fire resistance of exposed (i.e. unprotected) timber members.

3.2.5. CSA method

The Engineering Design in Wood guidance of the Canadian Standards Association (CSA) [66] employs a mechanics-based approach to verify the structural fire resistance of glulam columns and CLT slabs and panels at a specified time of fire exposure. The calculations relating to fire exposure of timber structural members given in CSA [66] are part of an informative annex. The charring rate for glued-laminated timber is given as 0.65 mm/min for one-dimensional charring. For CLT, it is suggested that the one-dimensional charring rate should be applied if the char front remains in the first lamella during the fire. If charring progresses into any subsequent layers, an average charring rate of 0.8 mm/min is recommended for the full fire resistance duration. Corner rounding can be accounted for via use of a nominal rate of charring of 0.7 mm/min for glued-laminated timber. For heated timber below the char, CSA recommends use of the RCSM (as outlined in EC5 [55]) with a ZSL of 7 mm.

To account for instability, a buckling reduction factor, k_c , is multiplied by the compressive strength, which as for EC5 [58] and NDS [43], depends on the element's slenderness ratio and strength-to-stiffness ratio. Fig. 2 shows the buckling reduction factor, k_c , for the different methods as described above. Shown are the factors that designers should apply to the compressive strength to account for the fact that structural elements

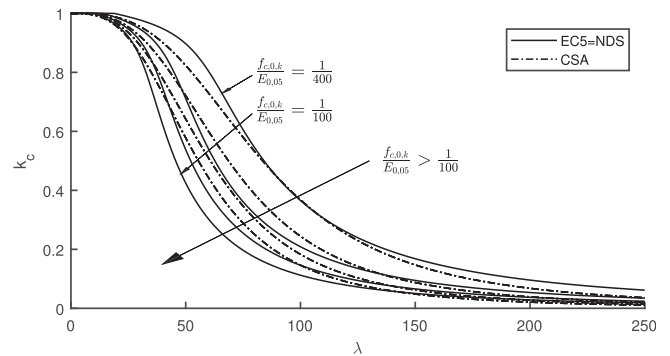


Fig. 2. Normalised strength reduction curves with slenderness and strength-to-stiffness ratios for laminated timber compression elements in fire for EC5, NDS and CSA. The CSA lines correspond to the same strength-to-stiffness ratios as the EC = NDS lines.

that experience instability cannot reach their theoretical compressive strength. It is clear that the applied reduction is greater for all methods with increased slenderness. Additionally, the EC5, NDS and CSA methods increase the reduction for increasing strength-to-stiffness ratios, since buckling depends on the stiffness of a member rather than its material strength, and a strong column with a low elastic modulus is more likely to experience instability. At low slenderness, failure by crushing is expected and k_c tends to 1.0.

Failure is then determined using a linear interaction equation between the compressive load capacity ratio and bending capacity moment ratio. For CLT protected with gypsum board, the CSA guidance suggests that the fire resistance rating can be increased by a nominal fixed value, depending on the thickness of encapsulation applied. For example, 12.7 mm of Type X gypsum board is assumed to increase the calculated fire resistance by 15 min; the interaction between encapsulation fall off and delayed and/or accelerated charring is not explicitly considered.

3.2.6. Stiller method

In 1983 Stiller [67] reviewed available calculation procedures from other German authors (e.g. Stanke et al. [25]) to predict the fire resistance of glulam columns, and concluded that a unified simple calculation method was required. To calculate the loss of cross section Stiller [67] empirically fitted a charring rate of 0.59 mm/min to measured standard fire test results to achieve the smallest possible deviations between his model and the observed failure times. Similarly to the RCSM, Stiller [67] proposes a heated zone of zero strength below the char layer; this is empirically fitted to be 6.4 mm based on results of tests by Stanke et al. [25] and Haksever [29]. For structural calculations Stiller [67] imposes a limiting buckling stress that includes an assumed eccentricity based on the chosen timber grade. These inherent eccentricities arise from imperfections in the timber and are assumed to be greater for lower grade timber. The resulting semi-empirical method calculates the fire resistance as the time when the char layer has progressed to a point where the acting stress (i.e. load over reduced cross sectional area) exceeds the limiting buckling stress.

3.3. Other methods not included in meta-analysis

3.3.1. Empirically-based methods

Malhotra and Rogowski [27] propose an empirically-based predictive formula (Eq. (6)) for fire resistance of glulam columns. This is based on 25 furnace tests on glued-laminated timber columns of varying species, shape, adhesive, and load level (which includes consideration of the slenderness and radius of gyration, to account for buckling). Based on a statistical analysis of their results, Malhotra and Rogowski [27] propose to predict fire resistance based on assigning experimentally-derived variables for each of their investigated parameters, and simply multiplying these variables in series to predict fire resistance times as follows.

$$t_f = T \cdot G \cdot S \cdot L \quad (6)$$

T , G , S , and L are factors related to species, adhesive type, shape, and load level, respectively. The proposed empirical value range for T is 2.06 (Cedar) to 2.64 (Douglas fir), for G it is 2.17 (Casein) to 2.64 (Phenolic), for S it is 2.04 ($b/d = 2.71$) to 2.64 ($b/d = 1$), and for L it is 2.64 (100% design load) to 5.28 (25%). For example, a phenolic glued Douglas fir column with a breadth-to-depth ratio of 1.0 and loaded to 25% of its ambient design load would have a predicted fire resistance of 97 min.

3.3.2. Semi-empirical methods

Scheer et al. [68] developed a thermal and structural finite element model, which they used to compute the load bearing capacity of timber columns of various dimensions after 30 min of standard fire exposure. They then developed and fitted Eq. (7) to their results to provide a straightforward means to calculate the remaining load bearing capacity (as a proportion of the ambient temperature capacity) for

glued-laminated columns for a target fire resistance of 30 min.

$$\eta_{c,0,fi} = \left(588 \frac{u^2}{A} - 58.5 \frac{u}{A} + 1.55 \right)^{k_{ref,c,0}} \quad (7)$$

A is the cross-sectional area, u is the circumference of the column considered, and $k_{ref,c,0}$ is an adjustment factor taken between 1 and 0.89 to account for the strength class used. This model cannot be assessed against the available data, as it can only be used to design columns for a target fire resistance of 30 min.

Stanke et al. [25], based on earlier work by Stanke [69], propose a semi-empirical equation (Eq. (8)) for the ‘critical fire resistance time’ t_f , which is based on a failure criterion linked to a critical slenderness, b_{cr} (in cm):

$$t_f = 7.2(b_0 - b_{cr}) + 1.56 \quad (8)$$

where b_0 is the width of the shorter column side and b_{cr} is the critical buckling width for this side. This critical buckling width is calculated by equating a limiting failure stress for instability with the acting stress during fire. During fire the acting stress increases due to a reduction of the uncharred loadbearing area, and the limiting stress for buckling reduces with increasing slenderness (due to charring and loss of cross-section). Ultimately the acting stress will exceed the buckling resistance and this gives the predicted critical buckling width. In Stanke et al.’s [25] papers this value is determined graphically at the intersection of the plots for the acting stress and limiting strength. In combination with the charring rate, this can be used to calculate the fire resistance (see Eq. (8)). Stanke et al.’s [25] model uses an empirically-fitted average constant charring rate of 0.695 mm/min, which does not begin until 1.6 min of fire exposure. The proposed model accounts for loss of mechanical properties of heated timber by simply assuming the next lower formal timber strength class, rather than the one assumed for ambient temperature design. This approach is problematic outside the jurisdiction for which Stanke et al. proposed their model (i.e. 1970s West Germany), since different jurisdictions apply different specific strength classes depending on a wide range of factors.

3.3.3. Analytical methods

Suzuki et al. [36] suggest to calculate the critical buckling load for a fire-exposed CLT wall using a conventional secant formula, which is derived from the Euler buckling equation, with the additional consideration of load eccentricity caused by the loss of effective cross section due to charring and in depth thermal gradients. To model the reduction of strength in heated timber specifically for CLT walls, Suzuki et al. [36] propose a simple effective cross-section analysis (not to be confused with the RCSM) wherein the widths of small horizontal slices of a CLT wall panel are reduced according to their current temperature and resulting local reduction in elastic modulus. In this approach, layers with different elastic moduli are scaled in width according to the ratio of elastic moduli, which are determined from linearly interpolated experimental in-depth temperature data, in conjunction with the reduction curve proposed by Kaku et al. [53] (refer to Fig. 1 b)). The critical buckling load and deflection of the wall can then be calculated using an analytically derived secant formula, shown in Eq. (9).

$$P_{cr} = \frac{\sigma_y A}{1 + e_c/r^2 \sec\left(\frac{\pi}{2} \sqrt{P_{cr}/P_e(T)}\right)} \quad (9)$$

where P_{cr} is the critical buckling load, σ_y is the compressive strength of timber, e and c are the eccentricity and location of the neutral axis from the edge of the remaining cross section respectively, r is the radius of gyration, and P_e is the Euler buckling load of the transformed (based on the temperature profile) section. Since the reviewed datasets for tests on CLT walls do not give information about the temperature profiles in the walls, this method cannot be included in the meta-analysis that follows.

3.4. Other issues

Due to the anisotropy of CLT resulting from its crosswise timber build-up, considerable differences exist between the strength and stiffness of layers oriented in the parallel and crosswise directions. This causes peculiarities for both ambient temperature design and for structural fire design of CLT elements. In most current design standards and guidelines, the direct influence of the weak layers on structural capacity is assumed to be negligible, and mechanical contributions from crosswise layers are generally ignored [70,71].

3.4.1. Shear deflections and rolling shear

Shear deformations, which can occur in CLT bending elements with a length to depth ratio below 20 [72,73], are usually more critical for verifying serviceability limit states, and they are therefore seldom considered in the fire limit state design of structural elements [74]. It should be considered, however, that increased deflections are likely to amplify the risk of instability failure modes that typically govern for timber compression elements in fire [25,27,33,36,54,67]. For compression elements under normal conditions only minor shear forces are likely to exist, and it is therefore assumed that shear deflections can be neglected in the current paper.

3.4.2. Structural boundary conditions for buckling of wall elements

In all reviewed test results for CLT walls, the tested elements were designed for uniaxial load carrying action, and the structural boundary conditions and resulting behaviour were therefore assumed to be analogous to the instability behaviour for columns. If additional restraints are provided to the sides of the walls, the buckling behaviour might be more analogous to that of a flat plate, since the two-way load bearing capacity of CLT could be exploited. The as yet unproven availability of demonstrably fire-safe connection details to assure two-way action in CLT walls is one obstacle to making use of two-way load transfer for slabs, although this might be less of an issue for CLT walls, which are usually limited in height and the length is only limited by available transportation and construction methods.

3.4.3. Non-standard fires

All of the test results and methods presented in this paper are based on standard fire exposures in fire testing furnaces, rather than real fires. The concept of fire resistance with units of time as measured in furnace tests was originally developed based on an assumed equivalent 'fire severity' – which was based on an equivalence between the integral of the temperature versus time curve – to inherently design for burnout of the compartment fuel load (as expected for different building occupancies) [75]. This original definition of fire resistance did not account for the potential additional contribution of combustible structural elements to the fire, since these were by definition outside the fire resistance framework (although the modern fire resistance framework applies different levels of required fire resistance notionally based on fire risk hazard, rather than the requirement for burnout). Clearly, exposed structural timber elements within a building may contribute additional fuel to a fire, so that a fire in a tall mass timber building with exposed timber structural elements can be expected to result in different, possibly more severe, fire dynamics to those historically assumed. Furthermore, it has been shown that the mean temperatures within timber elements can continue to rise throughout a fire's cooling phase [76]. These issues are not considered by any of the reviewed methodologies, and could compromise design solutions for exposed timber in practice, particularly in tall buildings.

4. Meta-analysis

Determining the specific mechanical properties of mass timber test specimens from test reports and available publications is, due to the inherent variability of timber's mechanical properties and the widely

used (but different amongst different jurisdictions) system of timber grading, challenging when the requisite mechanical properties are not explicitly stated in the assessed reports. In this section, the available furnace test data on mass timber compression elements are used to statistically compare the predictive performance of the respective fire resistance prediction/design models discussed in the preceding sections.

For the columns tested by Malhotra and Rogowski [27], the basic dry stress (where 'dry' refers to service conditions with moisture contents below 18%) is given for each timber species used, however no mention of the elastic modulus, which is critical to assess the buckling capacity of a member, is made. For the current paper for the results from these tests the elastic moduli were assumed based on CP112:1967 [74], which gives the mean values of the elastic modulus for different species groups and grades [77] that would have presumably been used by Malhotra and Rogowski at the time when their study was originally performed.

When using the permissible stress values for comparison against test data, it is also important to consider that the values may be quoted within different percentiles of a normal distribution. For example, modern strength classes typically use a 5th percentile value for strength in assigning the class [78], whereas CP 112:1967 used the 1st percentile [79]. Since the standard deviation and the mean in these historical datasets are not known, any comparison will necessarily contain inherent uncertainties.

Where no information is given in the source publications regarding the strength or elastic modulus of the timber used, these are inferred from the permissible stresses of the strength classes used in the country of origin at the time that the testing was undertaken, as outlined by EN 1912 [80] and taken from EN 338 [78]. Both Simpson [81] and Schmid et al. [60] have previously highlighted the significant challenges in obtaining and assessing historical data from standard fire tests (or any other fire or heated tests) on timber structural elements.

To obtain comparable results for all prediction methodologies, all input values for strength and elastic modulus were taken as mean values. These were either taken from reference tests quoted within the respective source documents, or back-calculated from the given characteristic values assuming the data to follow a lognormal distribution with a coefficient of variation of 15% for the strength and 13% for the elastic modulus of both glued-laminated and cross-laminated timber as recommended in the JCSS Probabilistic Model Code [82]. Codified design guidance will, in some cases, aim to achieve a certain level of conservativeness, rather than an accurate prediction of the fire resistance. This means that many of the methods presented and assessed herein contain inherent safety factors, and while explicit safety factors are omitted from the analysis there exist uncertainties associated with implicit safety factors from codified design guidance.

Calculations for CLT walls require consideration of axial load and moment interaction, and the assessment criterion for failure therefore require bending strength or bending moment resistance to be weighed against bending stresses or moments. In timber, bending strengths (also often called moduli of rupture (MOR)) are nominal values determined from failure at the tensile fibre in beam bending tests [68,83]. For CLT in fire, the theoretical bending resistance in fire varies between the compressive and tensile faces of walls in fire due to movements of the neutral axis and the changing elastic moduli of the various lamellae. Because bending strength values originate from tensile ruptures in beam tests, any bending resistance in the analysis presented herein is derived at the outermost tension fibre, which is assumed to lie on the unexposed side of walls (i.e. it is assumed that CLT walls under initially concentric loading can be expected to buckle away from their fire-exposed face).

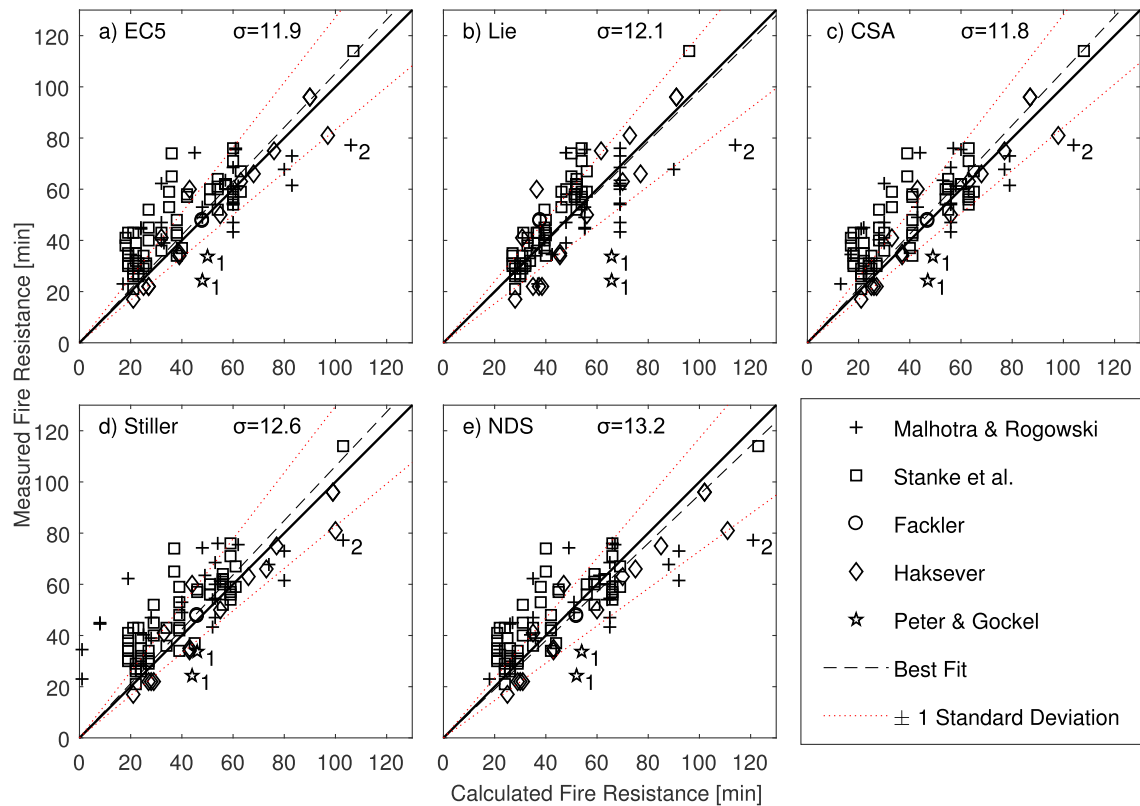
4.1. Glued-laminated columns

The main input parameters used for the meta-analysis for glued-laminated timber columns are shown in Table 1. The compiled data comparing measured and calculated fire resistances for all of the investigated prediction methodologies are shown in Fig. 3 for the glued-

Table 1

Key input parameters from datasets for furnace tests on glued-laminated columns.

Authors	# of Tests	Initial Slenderness Ratio λ	Effective Length [mm]	Mean Elastic Modulus [MPa]	Mean Compressive Strength [MPa]	P/P_u [%] ^b
Malhotra and Rogowski ^a [27]	28	19–51	2080	6900–11720	10–22	3–27
Stanke et al. [25]	56	31–105	3650	8630–15590	36–55	9–29
Fackler ^a [28]	2	44	2275	11000	30	9
Haksever [29]	15	51–80	2254–5910	14400	41	5–22
Peter and Göckel [26]	2	19	1200	11000–13000	37–42	4–5

^a Mean mechanical property values back calculated from reference data.^b P_u taken as Euler buckling load, P as applied load.**Fig. 3.** Meta-analysis plots comparing measured and predicted fire resistances for glued-laminated timber columns for: a) the EC5 method [55], b) Lie's method [54], c) the CSA method [25,66], d) Stiller's method [67], and e) the NDS method [43]. Standard deviation shown as σ . Fitted means are forced through the origin.

laminated column tests available in the literature. In addition to the individual data points, the mean linear fit to the data is displayed, as well as a line with a slope of unity that marks the boundary between conservative (points above) and non-conservative (points below) predictions. Finally, lines showing ± 1 standard deviations from the fitted mean are also shown for each method along with the standard deviation, to give a visual indication of the statistical uncertainties of each of the methods.

While design methods should generally return 'safe' design solutions, they should also not be overly conservative (i.e. inefficient), and should therefore be able to reasonably predict test results. When no safety factors (i.e. material or member reduction factors) are included in the calculations, the methods should ideally provide an accurate prediction of the unity line in Figs. 3 and 4, with as small a mean percentage error (MPE) and mean absolute percentage error (MAPE) as possible. MPE and MAPE were chosen as a relative measure of forecast accuracy between the assessed methods since the measured fire resistance data naturally does not contain zero or negative values and MAPE offers a simple, easy to read comparison for these cases [84]. One potential pitfall of MPE and MAPE as measures of forecast accuracy is that negative errors can be weighted heavier than positive errors, however, this is not a problem for

the comparison presented herein as all the measured fire resistance values are fixed and do not change between the assessed methods.

Assuming (admittedly somewhat arbitrarily) that the data points are normally distributed about the mean, the meta-analysis comparisons can be used to statistically quantify the performance and level of conservativeness of each of the assessed models in light of the available test data. Since all material input parameters used in the calculations were, to the extent possible, adjusted to reflect mean values, a 'perfect' model would have a mean slope of one and an MAPE of zero. This assumes that the assessed models aim to provide accurate prediction of the unity line. This is a reasonable approach because most modern codified guidance documents mostly base conservativeness on statistically defined structural reliability targets. The relevant statistical data for glued-laminated columns is shown in Table 2, which also gives the mean calculated loss of cross-section at failure for each assessed method. This can elucidate whether differences between models are caused by underlying assumptions for charring and heating, or rather by the assumed mechanics inherent in any given model. The statistical percentage number of conservative results is based on the distance in standard deviations between the unity line and the computed mean.

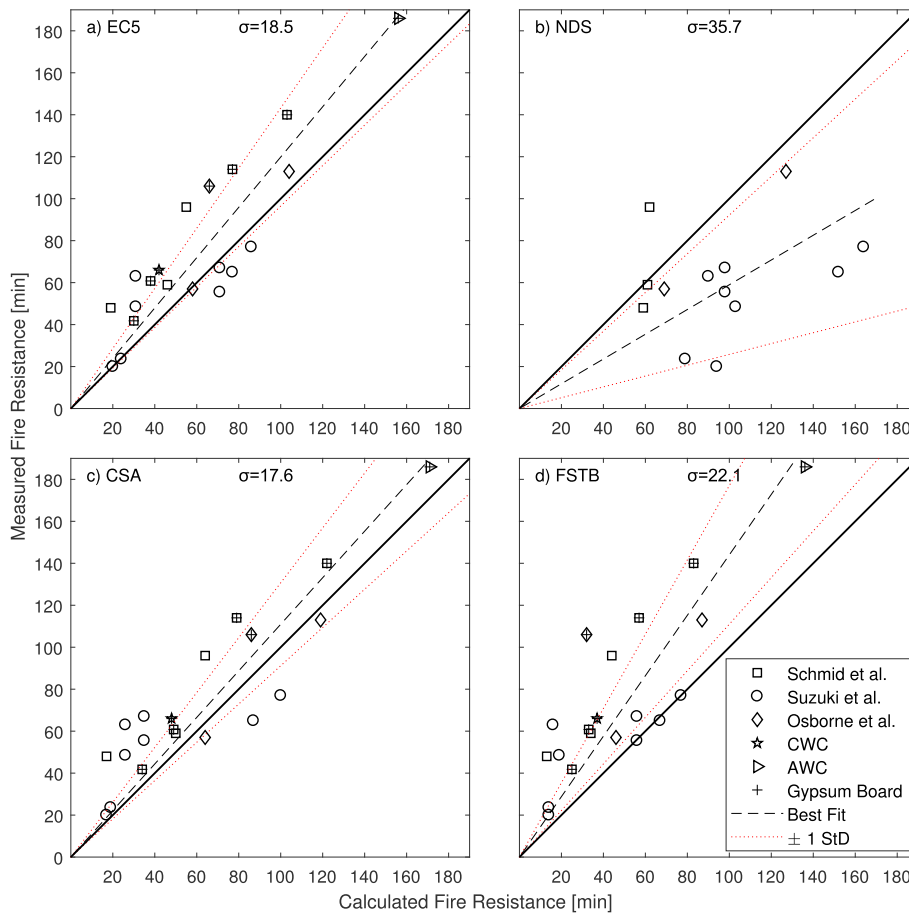


Fig. 4. Meta-analysis plots comparing measured and predicted fire resistances for CLT compression elements for: a) the EC5 method [55], b) the NDS method [43], c) the CSA method [66], and d) the charring and ZSL model as outlined in the FSTB [65] method (in combination with structural considerations in EC5). Fitted means are forced through the origin.

Table 2

Summary of meta-analysis comparisons for tests on glued-laminated timber columns.

Method	Best Fit Slope	MAPE [%]	MPE [%]	# of Conservative Results [%]	R^2 of Fit	Mean β_f at Failure ^a [mm/min]
EC5 [55,58]	1.03	22.4	10.4	58	0.50	0.87
Lie [54]	0.98	20.4	-4.2	46	0.57	N/A
CSA [66]	1.04	22.4	11.2	58	0.51	0.87
Stiller [67]	1.04	23.4	12.0	58	0.47	0.76
NDS [43,44]	0.93	21.9	0.6	38	0.49	0.79

^a Taken as the rate of loss of cross section (including heated timber at failure).

For glued-laminated columns, based on the parameters in Table 2, Lie's method [54] yields the most accurate predictions (having the lowest MAPE), despite relying only on load level, effective length and side dimensions, which are relatively simple input parameters. However, this might be due to the fact that Lie's method was calibrated against the majority of the reviewed test results available at the time of publication of Lie's model, and there are very few more recent test results against which the capacity outside of the empirically calibrated range can be assessed. Overall it appears that, for glued-laminated columns, all of the assessed methods provide reasonable and comparably accurate predictions of fire resistance on the basis of the available test data, however with a number of clear outliers in some cases.

One interesting observation in Table 2 is that all of the methods overestimate the fire resistance for the tests by Peter and Göckel [26]; these tests were on relatively stocky columns ($\lambda < 20$), and it thus seems that the reviewed methods may return non-conservative results for stocky columns with a low slenderness ratio. Most of the methods rely on a critical side length for buckling capacity calculations. In a very stocky

column the models may not capture failure since they are calibrated (either implicitly or explicitly) for buckling failures. Similarly, one result from the Malhotra and Rogowski [27] data set has a slenderness ratio below 20 and is predicted non-conservatively by all of the methods and identifiable as a clear outlier (lying outside one standard deviation from the mean) in all methods in Fig. 3. It should be noted that the methods by Stanke et al. [25] and Stiller [67] were derived using theoretical considerations and are unlikely to be in use today, while the other methods all have or had a perspective of use primarily for design.

A detailed analysis of obvious statistical outliers was undertaken in an effort to identify possible reasons for specific test results lying more than one to two standard deviations from the best-fit line; however, aside from the comments in the preceding paragraph, no obvious reasons or trends could be identified.

Based on the above results and discussion, it appears that a mean absolute error in the range of 20% is impossible to avoid using any model, whether physics-based or empirically derived.

Table 3
Key input parameters from datasets for furnace tests on CLT walls.

Authors	# of Tests	Initial Slenderness Ratio λ^a	Effective Length [mm]	Mean Elastic Modulus [MPa]	Mean Compressive Strength [MPa]	# of Lamellae	P/P_u [%] ^{b,c}
Schmid et al. [33]	7	49–78	2040–2470	9070–12320	43–46	3–5	7–22
Suzuki et al. [36]	8	43–49	2150–3300	4200–4670	25–31	5–7	14–48
Osborne et al. [34] ^a	3	53–88	3048	10190–12450	16–28	3–5	7–24
CWC [38] ^a	1	88	3048	9520	17	3	25
AWC [39] ^a	1	53	3048	12340	28	5	3

^a Mean mechanical property values back calculated from reference data.

^b Based on parallel layers only.

^c P_u taken as Euler buckling load, P as applied load.

Table 4
Summary of meta-analysis comparisons for CLT walls.

Method	Best Fit Slope	MAPE [%]	MPE [%]	# of Conservative Results [%]	R^2 of Fit	Mean β_f at Failure ¹ [mm/min]
EC5 [55,58]	1.2	25.1	18.4	79	0.76	0.92
NDS [43,44]	0.59	94.2	–88.8	12	–0.18	0.86
CSA [66]	1.11	27.7	19.6	73	0.74	0.82
FSTB [65]	1.44	37.8	37.4	90	0.58	1.24

^a Taken as the rate of loss of cross section (including heated timber at failure), for exposed timber tests only.

4.2. CLT walls

Fig. 4a), b), c), and d) compare the furnace test data gathered from furnace test results available in the literature for CLT walls against the Eurocode [55], NDS [43], CSA [66], and FSTB [65] methodologies, respectively. The FSTB methodology is essentially identical to the EC5 methodology; however it uses a slightly different approach to calculate the ZSL, as already noted. Since the NDS method is explicitly derived only for exposed (i.e. unprotected) timber members, only tests without any supplemental fire protection (i.e. no encapsulation) are included for this case. The main input parameters used for the meta-analysis of CLT wall furnace tests are given in Table 3.

The relevant statistical outcomes of the meta-analysis for CLT walls is given in Table 4 and using the same parameters as explained for glued-laminated timber columns above.

For CLT walls the EC5 and CSA methods result in similar MAPE values, as well as similar levels of conservativeness. While the accuracy of the EC5 method returns the lowest MAPE value (25%), the majority of the individual data points lie out with of one standard deviation from the mean, highlighting a level of inconsistency for this method (or, perhaps, for the test results themselves). A similar observation can be made for the CSA method, where most of the results from protected CLT walls lie within one standard deviation from the mean.

The alternative zero strength layer model proposed in the FSTB method yields more conservative results than the EC5 method; this is expected since the FSTB approach essentially increases the rate of charring (i.e. effective loss of cross section). The NDS methodology yields non-conservative results 88% of the time. The NDS methodology considers an increased charring rate for the fall off of lamellae, yet its assumed effective charring rate at calculated failure times β_{ef} are only slightly lower than for EC5. The reason for the limited predictive capability is therefore thought to be due to the mechanical aspects of the model. For instance, the nonlinear design interaction equation that is used to assess failure may not be appropriate. Indeed, Wang et al. [85] performed experiments on eccentrically loaded CLT walls and concluded that nonlinear interaction equations are non-conservative for some load cases for CLT, and should be replaced with a linear interaction equation (as is now used by the CSA guidance [66]). Additionally, the NDS uses a slenderness factor, c , of 0.9; however, Zahn and Rammer [86] state that this value was never validated and should probably be less. However, changing the slenderness factor in the NDS analysis does not significantly change the results, since its influence on the compressive strength of a

column or a wall is limited to $\sim 10\%$ (for a generic assumed relative slenderness). Additionally, the slenderness factor is not influenced by charring and its overall contribution to the design strength of a member also reduces with increasing fire duration.

Considering the results in Fig. 4, it is evident that the NDS methodology makes reasonable predictions for the results from Schmid et al. [33] and Osborne et al. [34], but is badly non-conservative for the tests by Suzuki et al. [36]. For the data from the latter, two differences can be identified as compared to the tests by the former. First, half of the experiments were performed on seven layer CLT (rather than five layer CLT). Second, the elastic modulus from reference test in Suzuki et al. is, on average, 37% less than what standards would recommend for the lowest strength grade in Europe [78]. With up to 48% (as a proportion of the theoretical ultimate buckling load) these tests imposed a higher load level than the other two test series. This suggests that the loading interaction equation in NDS should perhaps be revisited for 'high' loads at larger eccentricities, and for CLT with five or more lamellae.

The average MAPE for all methods applied to CLT walls is 46.2% (30.2% if NDS is excluded as an outlier) with a minimum MAPE of 25% and a maximum of 94%. The forecast accuracy is therefore less than for glued-laminated columns discussed in the previous section. This is not surprising since the structural and thermal boundary conditions for CLT walls are less well defined (e.g. fall off of layers) than for glue-laminated columns. In addition there are significantly fewer test results available for CLT walls compared to glulam columns. It is noteworthy that the calculations performed herein are based on mean input parameters, and while design methods should give safe results, they will lead to uneconomical designs if their predictive capability errs too far on the conservative side as a consequence of failing to capture the relevant physical realities. Structural engineering methods for CLT wall elements in fire at high loads (as could be expected as timber buildings increase in height) will need to be improved, and more reliable tools to produce economical and safe designs will need to be developed.

For the EC5 method [55], a reasonable correlation is observed for protected CLT walls in Fig. 4 a), suggesting that the current model for the delay of charring by gypsum board protection, and the subsequent increased charring rate once gypsum board falls off, may be adequate for the range of test results considered to date (although only seven data points are available). A similar observation can be made for protected tests assessed using the CSA [66] method.

5. Conclusions

A meta-analysis of available design methods to predict the fire resistance of glued-laminated timber columns in the context of the available furnace test results on this type of element has shown that the assessed methods predict the standard fire resistances with reasonable accuracy (average MAPE of 22.3%) and can therefore, with the addition of safety factors and use of characteristic values for timber properties, be utilised to design timber columns within reasonable, although currently un-quantified, error bounds. This suggests that the available models are able to reasonably account for the necessary parameters, either by appropriately accounting for the necessary physics, or by careful selection and calibration of appropriate empirically based equations.

The observed fire resistance forecast accuracy of the design methods for CLT walls was found to be relatively poor (when compared to the results for glulam columns). The average MAPE (excluding the NDS method as an extreme outlier) was found to be 30% between experimental observations and the prediction methods discussed in the current paper. The methodology proposed in EC5 [55], in combination with the charring model proposed by Klippel et al. [62], was found to yield the lowest MAPE with a value of 25%.

The alternative zero strength layer considerations in FSTB [65], a companion document to EC5 [55], was found to yield a statistically conservative outcome for 90% of the data investigated and to incur larger errors than the EC5 method. It is therefore questionable whether designers should prefer guidance given in FSTB over that from EC5, despite the former addressing CLT specifically. The methodology given by NDS [43] was shown to be statistically non-conservative in 88% of the considered cases. As the rate of loss of cross-section was similar for the assessed CLT wall models, the difference in the NDS calculation results can likely be attributed to the structural model used; this should therefore be revised, and its treatment of eccentric loading in particular reconsidered.

Due to an almost unique focus in the available data on standardised time temperature testing, no prediction methods were found in the literature that can be used by designers to accurately calculate the load bearing capacity of fire-exposed timber in the cooling phase of a fire or after extinguishment in a real-world scenario. This raises doubts as to whether the assessed methods are applicable for design of increasingly taller timber buildings with potentially different fire dynamics to those assumed by the dominant fire resistance design framework.

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