A REVIEW OF THE STATE-OF-THE-ART INTERNATIONAL GUIDELINES FOR SEISMIC DESIGN OF TIMBER STRUCTURES

V. Reale[1], S. Kaminski[2], A. Lawrence[3], D. N. Grant[4], M. Fragiacomo[5], M. Follesa[6], D. Casagrande[7]

[1] *Senior Engineer, Arup, vincenzo.reale@arup.com*
[2] *Associate Structural Engineer, Arup, sebastian.kaminski@arup.com*
[3] *Associate Director Structural Engineer, Arup, andrew.lawrence.@arup.com*
[4] *Associate Director Seismic Engineer, Arup, damian.grant@arup.com*
[5] *Full Professor, University of L'Aquila, massimo.fragiacomo@univaq.it*
[6] *Director Structural Engineer, dedaLEGNO, follesa@dedalegno.com*
[7] *Researcher, National Research Council of Italy, Institute of Bioeconomy, daniele.casagrande@ibe.cnr.it*

Abstract

The timber construction industry is expanding rapidly, fuelled by a growing awareness of the importance of providing sustainable buildings and the advantages timber buildings can offer off- and on-site, such as pre-fabrication and speed of assembly. While this market is also spreading into highly seismic areas in both developed and developing countries, research into the seismic behaviour of timber is relatively behind, compared with more conventional materials such as concrete and steel. Several national and international structural design codes (such as EC8, CSA-086, ASCE7 and ASCE41), now contain behaviour/response modification factors for timber that allow the designer to make use of dissipative zones in timber connections to provide ductility and allow deformations. However, the guidance is still generally very limited and primarily focused on light frame buildings using plywood/OSB shear wall systems, with little consideration of more modern systems such as heavy timber structures and CLT.

This paper provides an introduction to timber design in seismic areas, drawing on international codes, guidelines and published research to describe why timber has performed well historically, how a ductile response can be achieved in timber structures, and how one should approach seismic timber design. The paper then discusses the most common timber lateral load-resisting systems – light-frame, CLT, moment frame and braced frame – as well as a few less common ones and new technologies under development. Timber guidance in EC8, CSA-086, ASCE7 and ASCE41 is presented, and a few recommendations are made where gaps or potential areas of unconservatism in these codes were found. Because these codes are new and research is still limited in comparison with steel and masonry, structural engineers should supplement the guidance from these standards with other published research and general engineering and seismic best practice. The paper concludes with recommendations on modelling timber structures for seismic design, and areas of future research and possible code development.

Keywords: timber; standards; ductility

1. Timber as a structural material in seismic areas

1.1 Historical performance of timber buildings

Many traditional timber building systems have historically performed relatively well in earthquakes (Fig.1). This is clearly illustrated in areas with frequent large seismic events, where communities have developed effective vernacular solutions that have some inherent earthquake-resistant characteristics [1, 2, 3, 4].

The reasons for the positive seismic performance of historic timber buildings can be summarised as follows:

a) Timber has a high strength-to-weight ratio, therefore traditional timber buildings tend to be relatively light (compared to masonry and concrete), hence they are subjected to lower forces during earthquakes.

b) Traditional timber connection systems, such as nails or traditional carpentry connections where the primary load path is compression (parallel or perpendicular to grain), can usually resist relatively large deformations without failing, and some of them can absorb limited amounts of energy through plastic hinges in the nails, friction and local damage [5].

c) Unlike many traditional unreinforced masonry buildings, traditional timber buildings generally have a degree of tying, which helps to distribute the forces and ensure building components move together.

d) Since traditional timber buildings tend to be relatively light, their collapse is less of a life safety risk to the occupants. In addition, repairs may be easier.

Fig. 1 – Traditional timber buildings after the 2016 Kumamoto Japan earthquake [6] and 2016 Muisne Ecuador earthquake [7]

1.2 Modern timber buildings

Nowadays timber is becoming increasingly utilised in the building industry as a structural material, gradually becoming a competitive alternative to other construction systems [8]. Modern buildings are required to have reliable strength and tight deflection limits (due to glazing and finishes), therefore many of the seismic benefits associated with traditional vernacular constructions discussed in **1.1** cannot be exploited.

The reasons behind the recent increased use of timber in the construction industry are predominantly: its good mechanical characteristics, its good strength-to-weight ratio (which is comparable to steel and much better than concrete), the possibility to use engineered wood products (e.g. glulam, cross-laminated timber (CLT)), the advantages in off-site and on-site construction such as speed, and its positive sustainability credentials [8].

1.3 Properties of timber and timber connections under seismic loads

Timber as a material is inherently brittle. When subjected to bending, tension and shear it generally exhibits sudden and brittle failure modes, possessing limited ductility only when subjected monotonically to compression parallel or perpendicular to the grain [9].

To achieve ductility, which is key for the design of structures in seismic areas, the most common strategy is to make use of ductile steel fasteners (nails, screws, bolts and dowels) in timber connections. These fasteners can reliably dissipate energy under reversible cyclic loading by forming plastic hinges with good hysteresis behaviour (Fig.2, Fig.3, [10]). A small amount of energy is also absorbed due to local crushing of the timber, but this is not considered to be the primary source of ductility in these connections, and mainly contributes during the first cycle. Note that all steel connections in timber exhibit ductility only when they are subjected to shear forces (i.e. the fasteners are put into local bending), while in tension they are very brittle and hence codes do not permit their use in tension in dissipative zones.

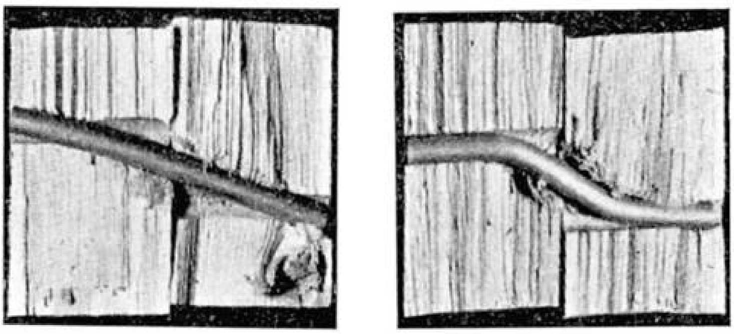

Fig. 2 – Images of nail failing in different ways in timber [10]

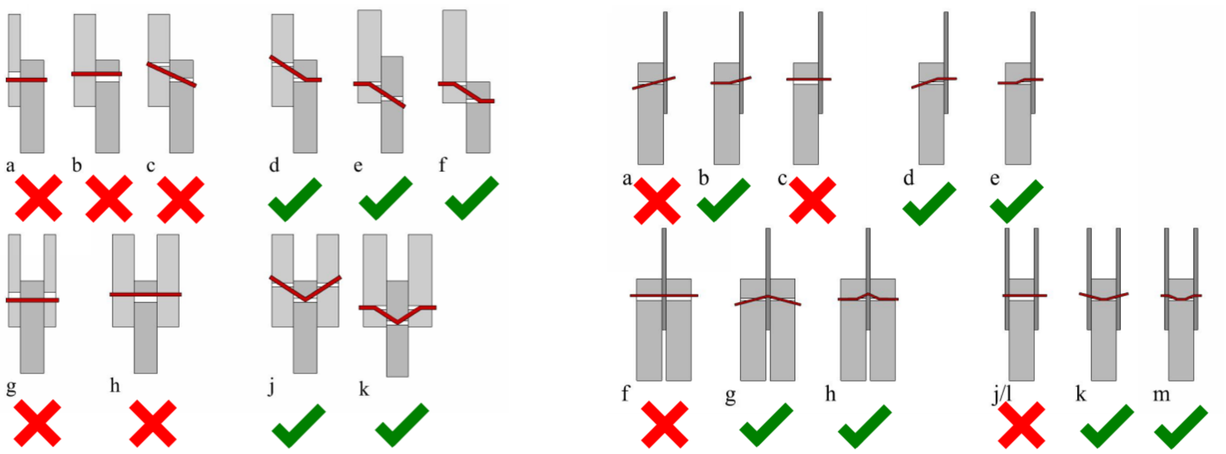

Fig. 3 – Johansen failure modes for timber-to-timber and timber-to-steel, showing which are considered ductile

Different connection systems have different ductility depending on if they use metal fasteners, the type of fastener used, and if they are designed for reversible cyclic loading (i.e. connections that have a relatively symmetrical hysteresis). In general and when properly designed [11, 12]:

a) Timber-to-timber connections with fasteners that are designed for reversible cyclic loading are the most ductile (e.g. plywood-to-timber nailed connections, timber-to-timber moment frame bolted connections). This is primarily because the risk of a shear failure within the fastener is lower since the local crushing of the timber leads to a more gentle/gradual deformed shape of the fastener. Together with the yielding of the fastener, there may also be a little additional ductility provided by the crushing of the timber around the fastener. These connections are most ductile when two plastic hinges form.

b) Timber-to-steel connections with fasteners that are designed for reversible cyclic loading demonstrate some ductility (e.g. dowelled flitch plate connections in moment frames), but generally less than typical timber-to-timber connections, because the risk of a shear failure within the fastener is slightly higher than timber-to-timber connections. These connections are most ductile when two plastic hinges form.

c) Timber-to-timber connections that are not designed for reversible cyclic loading demonstrate limited cyclic ductility (e.g. timber-to-timber carpentry connections). This is because they rely upon local crushing of the timber perpendicular to grain for limited ductility, and this mainly contributes during the first cycle.

d) Timber-to-steel connections with fasteners that are not designed for reversible cyclic loading demonstrate limited cyclic ductility (e.g. some CLT tension tie-downs). This tends to be because their relative flexibility increases their yielding displacement and makes it harder for them to attract load in load reversal. In addition, after displacement in one direction the component may buckle upon load reversal. Finally, nails in these connections can be susceptible to head-tear off.

In addition to the above, different fastener types have different ductility capacities as follows:

a) Smooth nails are generally very ductile connectors because:

a1) They have good cyclic behaviour.

a2) Plastic hinges are easier to form in them (because they tend to be slender, they have a low bending strength compared to their shear strength, and therefore the risk of a brittle failure mode is lower).

a3) Their withdrawal strength is low (compared to ring shanked etc) as they tend to pull-out slightly under shear. Tests have shown that screws and angular ringed shank nails in steel-to-timber dissipative connections may be at risk of fastener head shear off during cyclic tests [13], hence smooth nails can be preferable. Note that when using smooth nails there is a risk of separation of the two elements under large displacements – this can be mitigated by adopting longer nails.

b) Bolts and dowels can also be very ductile for the same reasons as smooth nails, and are usually made of lower strength hence more ductile steel. However, because of their larger diameter, if their length is short it can be more difficult for plastic hinges to form in them before a brittle failure mode.

c) Screws can vary significantly in ductility. One assessment [14] showed that smaller diameter screws with the same steel grade and from the same manufacturer have less ductility than larger diameter screws. The authors can only hypothesise that this might be because:

c1) The rope effect contribution (which reduces the ability for the fastener to "slip") is larger in smaller diameter screws (and slipping may assists in the formation of a plastic hinge).

c2) Smaller diameter screws have a very low torsional constant J and therefore experience higher torsional stresses than larger screws during insertion. The manufacturers could be compensating for this by increasing the tensile strength of the smaller diameter screws, however a higher strength normally leads to a lower ductility capacity.

1.4 General approach for seismic design of buildings

Once an appropriate lateral load-resisting system is selected, an estimation of its ductility should be carried out. Parts of the system with the ability to dissipate energy through deformation (ductile parts) will allow for the use of a reduced acceleration design spectrum, thus reducing the seismic design forces. At the same time, brittle parts of the system will have to be designed with enough strength to resist the displacements and demands allowed by the ductile dissipative components. Note that when designing steel plates, it should be ensured that the only dissipative zones are those where fasteners fail flexurally.

To date, the main timber lateral load-resisting system used in seismic areas has been light frame timber, which generally performed well (Fig.4, [15]). Over the past 10 years there has been a significant increase in interest for the implementation of timber in the construction industry worldwide, including in seismic regions, which has led to new research such as the development of new systems and major planned updates to US, European, Canadian, Japanese and New Zealand standards.

Because research into the seismic behaviour of timber is still relatively new, and gaps still exist in the published and codified literature, designers are recommended to always apply first principles thinking and good practice. In many cases it may be useful to consider guidance from multiple codes, ensuring compatibility among the relevant assumptions and following all local legal requirements. Where different codes or guidance provide contradictory information, it is suggested that the most conservative requirements are adopted.

Due to limited historical data on earthquake performance of modern timber buildings, difficulties in achieving drift limits and the very high overturning forces, most codes currently limit timber building heights to 20-30m in highly seismic areas [16, 17]. Even without these code limits, it is currently practically very challenging to design timber buildings higher than this for the aforementioned reasons.

Fig. 4 – Light frame timber buildings after the 2016 Kumamoto Japan earthquake [6]

2. Timber lateral load-resisting systems for seismic areas

2.1 Common systems

The following are the most common timber structural systems which have been used and are codified for design in seismic areas. Their properties are summarised in Table 1.

a) Light-frame timber buildings. These are composed of horizontal and vertical timber elements, generally made of solid wood sheathed with panel products, typically plywood or oriented strand boards (OSB), or less frequently gypsum fibreboards.

b) CLT buildings (Fig.5). These rely on a series of CLT walls for lateral stability. Typically, these systems also consist of CLT floors and roof diaphragms.

c) Moment-framed timber buildings. These resists horizontal forces using structural portals, relying on semi-rigid connections between members. To obtain this type of connection, steel flitch plates fixed with dowel-type fasteners are generally provided at the connections between timber members. Note that fully rigid moment connections in timber are nearly impossible to achieve in practice, mainly due to inevitable slip of the mechanical fasteners.

d) Braced-timber buildings (Fig.5). These are generally composed of timber columns and beams which are pin-connected through mechanical joints and braced by timber elements. They are relatively flexible when compared to steel and concrete braced frames, however they provide greater stiffness than moment-resisting frames and therefore lower displacements.

Fig. 5 – CLT building (left) and heavy timber braced frame (right – permission from Structurlam)

Table 1 – Summary of key properties for different timber systems [15, 16, 22, 23]

Structural system	**Historical earthquake performance**	**Level of research**	**Ductile components**	**Potential ductility**	**Relative elastic load capacity**	**Building types most suited to**	**Height limits (current codes)**
Light-frame	Significant – good performance	Some	Panel to frame nails	High	Moderate	Residential	20 m (ASCE7)
CLT	Little/none	Some	Panel-to-panel screws, tension tie-downs	Moderate	High	Residential, offices, commercial	20-30 m (CSA O86:19)
Moment frame	Little/none	Little	Moment connection fasteners	Moderate	Low	Single storey roofs	15-20 m (NBC15)
Braced frame	Little/none	Little	Bracing elements / Bracing connections	Low	Moderate	Residential, offices, commercial	15-20 m (NBC15)

2.2 Less common systems

There are also some other less common timber structural systems which possess some ductility capacity. The new draft of Eurocode 8 as described in [18] includes some of these: log house buildings, framed walls with carpentry connections and masonry infill, timber trusses and vertical cantilever systems. Guidance for seismic design of glulam arches is provided in [19].

2.3 New technologies under development

One alternative system that has been recently explored, mainly in the United States, it is the Heavy Timber Buckling Restrained Braced Frame (HT-BRBF). This system consists of a steel frame embedded within a glulam or CLT casing. The timber encasement provides buckling restraint to the steel, which is capable of sustaining inelastic deformations in tension and in compression. Initial studies [20] have shown that this system exhibits promising properties, such as a significantly greater total energy dissipation compared to the conventional timber bracing systems.

Another new system is the proprietary Pres-Lam ("prestressed laminated") which consists of high strength steel cables or bars embedded between timber beams and columns or timber shear walls. In the case of timber shear walls, the prestressed cables allow for the re-centering of the walls after rocking, while the reinforcement bars connecting the walls and foundations provide energy dissipation through axial yielding [21]. This system has been successfully implemented in a few low-rise developments in New Zealand.

3. Specific codes recommendations

Major updates are currently underway in national timber construction standards around the world. Table 2 summarises some of the latest codes which contains seismic guidance for timber buildings. In the following sections further details regarding the design with some of these standards is provided. More information on design provisions for seismic design of CLT in Japan, New Zealand, Chile and China is provided in [17].

Table 2 – Summary of seismic codes containing timber provision

Code Name	**Country**	**Current published version**	**Upcoming revision**	**Seismic Guidance**				
				Light-Frame	CLT	Moment frame	Braced frame	Other Systems
EN1998 [24]	Europe	2004	2025	Y	UR	Y	Y	Y
CSA-086 [22]	Canada	2019	NP	Y	Y	N	Y*	N
ASCE7-16 [16]	U.S.	2016	2020?	Y	UR	N	N	N
ASCE41-17 [25]	U.S.	2017	2020?	Y	N	N	N	N
NZS3604 [26]	New Zealand	1999	2020	Y	N	N	N	N
BSL [27]	Japan	2019	NP	Y	Y	Y	Y	Y*

NP: not planned   UR: contained in upcoming revision   *limited guidance published

3.1 Designing with Eurocodes

The provisions for seismic design of timber buildings in the Eurocodes are currently contained in Section 8 of [24]. This relatively short section, mostly with only high-level rules for timber design in seismic areas, is currently under review together with the rest of the seismic code. Many of the key provisions and principles of this review have been published in [18]. Since then, a further review has been carried out and a draft is due to be published in 2025.

The approach described in the new Eurocode draft is founded on recent extensive research. Overall, the authors consider all the provisions to be reasonable and achievable. The draft includes the most common, and some relatively uncommon, typologies for timber buildings.

The overstrength factor defining the minimum capacity ratio between ductile and non-ductile elements within a system (e.g. capacity of timber panels versus capacity of ductile connectors) is generally considered as 1.6. This factor has been calculated both via direct assessments (extensive component and full-scale building testing), and via probabilistic assessments based on initial test data. When considering connections, the draft code suggests that the ratio between non-ductile and ductile failure modes for the same fastener should be greater than or equal to 1.2.

New rules for the design of plywood and CLT shear wall systems will be introduced. The new approach implemented follows closely what is described in [28], which could be used as a design guide until the official publication of the code.

3.2 Designing with Canadian standards

Seismic provisions for buildings in Canada are currently contained in the National Building Code of Canada (NBC) [23] in Section 4.1.8. Behaviour factors for timber structures are provided, but only for light-frames with shear walls sheathed in wood or gypsum panels. Specific guidance on the design of timber building in seismic areas is further provided in the recently published CSA-O86 [22]. Section 11 of this document contains provisions for light-frame and CLT buildings, including guidance on ductility and capacity design. The NBC is currently under a process of review and a new version is due in 2020. In most cases, provisions of the current NBC and CSA-O86 are very much aligned with the draft European ones.

The behaviour factors in the current NBC are similar to the draft Eurocode's ones, with the only exception of braced systems, which are provided with higher response modification factors, however no guidance is provided on what details or capacity design principles are required. In the absence of further guidance, it is recommended that the more conservative Eurocode values and design guidance are adopted.

Further guidance on the practical application of the code provisions together with worked out examples, specifically for plywood and CLT shear walls, is provided in the Canadian Wood Council Wood Design Manual [29], Section 8. The approach here described is very similar to [28].

3.3 Designing new buildings with US standards

In the United States, ASCE 7 – Minimum design Loads and Associated Criteria for Buildings and Other Structures [16] contains the general seismic design criteria for the design of new buildings. While Section 11 provides general seismic design criteria, Section 12 contains specific requirements for different structural systems and a table with the behaviour (here referred to as "response modification") and overstrength factors applicable to specific elements of the building (e.g. "collectors" and foundations). The material specific codes are then intended to specify the capacity design rules appropriate for each of the systems outlined within ASCE 7.

With regards to timber, ASCE 7 includes response modification factors only for sheathed light frame shear walls. For all other timber systems, the code requires a response modification factor of 1.5 to be used, effectively designing the structure as elastic. It should be noted that US codes allow for the demonstration of compliance by "alternative means", enabling the industry to include new systems and possibly seismic provisions not within the code framework [30] (although this triggers many additional permitting requirements which can be onerous).

The current material specific code for timber in the US is the National Design Specification (NDS) for Wood Construction (2015) [31]. The NDS provides guidance solely on timber, glulam, CLT and their connections, but no information on seismic design. Guidance on designing light frame timber buildings in seismic areas can be found in the Special Design Provisions for Wind & Seismic (SDPWS) (2015) [32], which are complementary to the NDS. Guidance on designing CLT timber buildings in seismic areas can be found in the CLT Handbook published by FPInnovations [33], although it is not up to date with the latest NDS. A new version of the SDPWS is due to be published in 2021, and the NDS in 2024.

While designing with US standards, it is worth noting that the response modification factors are greater than the values in the European codes. This suggests that a building designed following these standards will likely suffer more damage in an earthquake than a similar building designed to Eurocode [34]. This is a general observation that applies to most structural types.

In all these standards, it is important to note that the overall seismic design provisions for timber, as well as capacity design rules, are not as detailed as in the European and Canadian Standards. It is also worth noting that neither the current nor the latest NDS cover a number of key checks such as fastener group effects (specifically block and plug shear (Eurocode 5 Annex A [35]), splitting perpendicular to the grain (Eurocode 5 cl. 8.1.4 [35]), shear failure (Manual for the design of timber building structures to Eurocode 5, Section 6.2.4.5 [36])). In comparison to Eurocode, they also allow very closely-spaced fasteners with little reduction for group reduction effects.

Accordingly, when following US codes for seismic timber design it is recommended to supplement capacity design rules and connection provisions with Eurocode 5 [35] and Eurocode 8 [24] guidance. In particular, it is recommended that the more conservative Eurocode 5 rules are followed for fastener spacing, fastener group effect reduction factors and group failure mode checks.

3.4 Assessing and retrofitting existing buildings with US standards

In the United States, ASCE 41 – Seismic Evaluation and Retrofit of Existing Buildings [25], contains the general seismic criteria for the assessment and retrofit of existing buildings. Instead of response modification factors (R), component capacity modification factors (m) are used which apply to specific elements rather than to the system as a whole. The philosophy behind this method is to reduce the amount of retrofitting required by reducing unnecessary conservatism, accepting higher levels of damage compared to new buildings and reducing the material factors of safety. Broadly speaking, ductile components of a structure are referred to as deformation-controlled, while brittle components are considered force-controlled, depending on the exact force-displacement curve and whether the element is a primary or secondary structural element. The factors and provisions for timber structures are included in Section 12, although only for light-frame buildings.

The m-factors provided in the code for most timber components aligns with other research, and there is generally adequate information with which to assess and retrofit simple light-frame buildings. There are two key areas which the authors have observations on.

The code allows the designer to assume that timber frame components in axial tension and/or bending, timber piles in bending and axial, cantilever pole structures in bending and axial, and pole structures with diagonal bracing, are deformation controlled. As previously mentioned, timber is generally considered to be very brittle in tension and bending, and the three structures mentioned can feasibly have brittle failure modes, even if some of their components could be considered deformation-controlled. The authors assume that a “component” ductility is being conflated with “system” ductility (for example a pole structure with diagonal bracing can potentially have some ductility, likely at the connection, relying upon elongation of the steel or local rushing of the timber, however there are also many brittle failure modes in this system). It is therefore recommended that the designer uses a more conservative approach which still follows the minimum code requirements, and considers all of the following timber failure modes as force-controlled: axial tension parallel and perpendicular to fibre, axial compression parallel to fibre, bending, shear. Axial compression perpendicular to the fibre can be considered as deformation controlled. Furthermore, frame components, wood piles, cantilever pole structures and pole structures with diagonal bracing should not be considered ductile “systems” as a whole, but each of their components should be individually checked.

Regarding connections, the code simplistically considers all individual connectors to be ductile, however considers demands on bodies of connections (e.g. splitting in connected timber element or steel tie-downs) are force-controlled (cl. 12.3.3.1). In practice, individual fasteners can also develop brittle failures (e.g. shear of the fastener head), and brittle fastener group effects normally govern regardless. It is therefore recommended that brittle failures of the fasteners are all assessed as force-controlled, and that group failure modes (block and plug shear ([35] Annex A), splitting perpendicular to the grain ([35] cl. 8.1.4), and shear failure ([36] Section 6.2.4.5)) are checked first before checking individual fasteners

As already noted in **3.3**. it is also recommended that Eurocode 5 [35] rules are followed for fastener spacing, fastener group effect reduction factors and group failure mode checks.

4.0 Modelling and analysis

Modelling and analysis of timber structures is typically more challenging than other materials such as steel and concrete because:

a) Timber connections have a high variability in stiffness [12].

b) Timber and its connections are more prone to brittle failure modes.

The positive historic performance of timber structures can be partially attributed to designers deriving load paths from experience, based on the stiffest path, and the use of ductile and robust systems such as light-frame walls which also tend to have lots of redundancy. Based on the authors' observations, nowadays many designers tend to heavily rely upon computer software to assign load paths that may not represent reality, without checking them by hand from first principles, using a material (timber) which typically does not allow much ductile load redistribution within the system. In addition, modern timber lateral load-stability systems such as CLT and braced timber frames often have less redundancy, as buildings are larger and clients and architects want to reduce internal walls and bracing. Timber buildings do fail as a result [37]. In seismic areas, the derivation of the actual load path (or paths) is even more important, so that specific parts of the structure can be designed to fail in a controllable way.

There exists little published guidance for modelling of timber structures in seismic areas. The draft version of Eurocode 8 will provide some useful guidance, in particular the following:

a) Consider the stiffness of semi-rigid joints, in particular when calculating the natural period of vibration of the building.

b) Upper bound limits are provided on the natural period of vibration, because simplified numerical models can easily produce unrealistically high and therefore non-conservative natural vibration periods.

In addition to the above, the following steps are recommended for modelling of timber in seismic areas, regardless of whether the load path is being derived by hand or using computer models:

a) Start the design with the definition of a sensible load-path and lateral load-resisting system (rather than finding out from the computer model what the load path might be). Determinate systems are preferable.

b) For each connection, consider the range of stiffnesses it might have.

c) Compare the fundamental period derived from the computer model with a simple hand calc (e.g. following eq. 4.6 EN 1998). If the two differ significantly, the model may not be accurate.

d) Determine a base load path based on the most likely (average) stiffnesses of connections and elements.

e) Conduct a rigorous sensitivity analysis, varying the different connection and element stiffnesses within their realistic bounds. Combinations of upper and lower bound values should be checked, however use engineering judgement to reduce the number of permutations.

f) Brittle elements should be designed with appropriate overstrength factors for the envelope of the load paths based on all the different sensitivity analyses carried out.

5. Recommendations for areas of future research in seismic design of timber

The following areas are currently lacking significant or complete information in the field of seismic timber design, and future research would be very useful in order to further develop codes:

a) Further testing to derive behaviour/response modification factors for light-frame, moment-resisting and braced frame systems.

b) Appropriate height limits for timber construction.

c) Behaviour of opening in light-frame and CLT walls and diaphragms.

d) Rules of thumb for deriving the fundamental period of different timber structures (light-frame, CLT, moment-resisting and braced frame systems) as a way of checking computer analysis.

e) Stiffnesses of connections for modelling purposes.

6. Summary

In summary, there is significant potential for greater use of timber for buildings in seismic areas. To date, experience in real events has been largely limited to light-frame buildings; however, with recent research now being incorporated into seismic codes, designers can reliably design buildings incorporating CLT, braced frames, moment frames and other timber systems. Because these codes are new and research is still limited in comparison with steel and masonry, structural engineers should supplement the guidance from these standards with other published research and general engineering and seismic best practice.

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