WORLD CONFERENCE ON TIMBER ENGINEERING 2025
BRISBANE, AUSTRALIA
Advancing Timber for the Future Built Environment

SEISMIC DESIGN OF TIMBER BUILDINGS IN NZ - THE NEW NZS AS 1720.1

Tobias Smith[1], Daniel Moroder[1]

ABSTRACT: This paper describes and reviews the new building code requirements for seismic design of timber buildings in New Zealand. The new design requirements are based on a hierarchy of seismic design, allowing the choice of varying levels of ductility, ranging from elastic design to nominally ductile, limited ductile, or fully ductile structures. A capacity design procedure is then used to ensure that ductility occurs in certain selected Potential Ductile Elements (PDEs), while more vulnerable elements known as Capacity Protected Elements (CPEs), are protected by overstrength calculations. To ensure an acceptable margin remains between the expected performance of the PDE and eventual loss of support a requirement to confirm local in-elastic displacement demand is also included in the standard which requires an engineer to convert their target global ductility into a local demand while accounting for elastic displacements within the CPE.

The paper describes the new design process as used in New Zealand, its uniqueness as a performance-based design method, several of the efforts that were made in the standard to improve seismic resistance and identifies significant gaps in knowledge where future research is required.

KEYWORDS: Timber Engineering, Seismic Design, Standards, Building Regulation, Practicing Engineers

1 – INTRODUCTION

In 2022 a new standard for the design of timber-based structures was published in New Zealand. NZS AS 1720.1:2022 [13] replaced NZS 3603:1993 [12], and mainly consists of an adoption of the current Australian design standard AS 1720.1:2010 Timber structures Part 1: Design methods [8]. Two new chapters address the fact that New Zealand has a significantly higher seismic hazard compared to Australia; these are Chapter ZZ9 Seismic Design and Chapter ZZ10 Floor and Roof Diaphragms. The authors of this paper led the writing of these two chapters with the assistance of researchers and other practicing engineers.

A wide range of current international standards were consulted along with recent local and global research. The challenges faced were the use of performance based seismic design, key to the way New Zealand partitioners respond to the country's extreme seismic hazard, when most international standards take a more prescriptive response to seismic design (refer to draft EC8 [3], CSA O86-24 [1]).

This decision meant the standards committee were required to define several new terms and processes to be undertaking the design of a timber structure that, while not unique to the world of seismic design, are unique to the design of timber.

2 – THE PERFORMANCE BASED SEISMIC DESIGN PROCESS

Once the NZS AS 1720.1 is cited in the New Zealand Building Code (NZBC) [5], the seismic design chapters of standard give the minimum seismic design requirements for timber structures under the Verification Method of the NZBC. While alternative ways of compliance can be used, the verification method B1/VM1 is most often used for all buildings other than 1 and 2 storey residential dwellings.

In New Zealand, performance-based standards enable a designer to choose how they want a building to respond to seismic action, and to demonstrate how it will perform, based on the designer's selection of ductility. In comparison to more prescriptive design standards, NZS AS 1720.1 does not assign an assumed performance to a specific structural system. For example, while Light Timber Frame (LTF) shear walls are known to have a moderate to high level of ductile capacity, a designer may also choose to classify a wall

[1] PTL Structural & Fire, Christchurch, New Zealand

https://doi.org/10.52202/080513-0405

as either nominally ductile or elastic. While these classifications increase the level of seismic demand on the wall, they lower the need to provide ductility and use capacity design reducing design time and potentially cost.

The use of performance based design often means the engineer selects performance targets and then has to show their selected system meets those targets, requiring re-design if this is not the case. The use of performance based design as applied in NZS AS 1720.1 is split into three specific stages:

1. The engineer will select the Lateral Load Resisting System (LLRS) and the target drift
2. If the engineer has chosen to lower seismic demand on the LLRS using ductility, they will identify which elements in the LLRS will provide ductility and which elements are unable to provide ductility and need to be capacity protected
3. The engineer will then confirm the ductile elements do not exceed their inelastic deformation capacity

Steps 1 and 2 in the above list are common for all building materials within New Zealand's performance based design framework. Where timber differs however, is that while within concrete and steel design the subsequent materials standards provide clear requirements on how to achieve the required deformation capacity, this was not deemed as straight forward in timber.

The main reason for this is that while in timber structures typically it is some form of metal fastener that provides ductility, this fastener sits within a wide range of structural systems, from light timber framed shear walls to timber cross braces and moment frames. Furthermore, the type of fastener within the specific system can vary, for example nails or screws within a shear wall or bolts or screws in a gusset plate. The location of target ductility can also differ, for example the hold downs or panel joins, or both mechanisms, in a CLT shear wall.

Added to this, many timber systems have a high amount of inherent elastic deformation which can significantly limit the system's ability to mobilise ductile response.

3 - SEISMIC CLASSIFICATION AND THE DISPLACEMENT WINDOW

Designers can select the seismic classification for the design of any building. This classification establishes the global ductile capacity a designer expects their structure to have and the subsequent Structural Ductility Factor, μ. To calculate base shear the engineer will reduce the elastic site specific earthquake spectra as a function of μ in order to calculate the inelastic site demand on the structure.

The classifications of structure are as follows:

- Elastic structures are those containing primary seismic resisting members or connections that are not capable of (or not designed for) inelastic deformation. Elastic structures do not require capacity design. Elastic structures have a structural ductility factor of $\mu = 1$ and therefore are designed using the elastic site specific spectra. Brittle structures are a subset of elastic structures which are further discussed in Section 5.2.
- Nominally ductile structures are those capable of sustaining small inelastic material deformations. Capacity design must be used although this is often capped at the elastic site specific spectra as described further in Section 5.3 below. Nominally ductile structures have a structural ductility factor of μ between 1 and 1.25.
- Limited ductile structures are those capable of ductile behaviour under seismic loading greater than their design capacity. Significant inelastic deformations are required in ductile elements to limit earthquake design forces. Limited ductile structures have a structural ductility factor of μ between 1.26 and 3.
- Fully ductile structures have a structural ductility factor of greater than 3 and are excluded from the verification method. This requires an alternative solution to compliance and are typically the subject of a special study.

The ductility selected is used to determine the inelastic site spectrum scaling factor, k_{μ}, which is applied to the elastic site spectrum. Additionally, the Structural Performance Factor, S_p, is used to reduce the elastic spectrum accounting for effects outside of the impact of ductility that are not easily captured such as peak versus average response and the typical underestimation of total capacity and actual damping (for example damping from non-structural elements) [10].

3.1 LIMITATION OF FULLY DUCTILE STRUCTURES IN NZS AS 1720.1

During the writing of the seismic chapter of NZS AS 1720.1 a decision was made to exclude structures that target a global ductility higher than 3. This does not eliminate the possibility of their design in New Zealand but does mean that design for a ductility of greater than three becomes an 'alternative solution' where compliance is shown on a case-by-case basis. The motivation behind this decision is the realisation that achieving high levels of ductility in what are typically flexible timber systems is challenging as they often have a small 'displacement window' where in-elastic response in possible.

The 'displacement window' is the difference in lateral displacement between the onset of yielding and the upper bound prescribed by codes to maintain structural stability. The larger the displacement window, the easier it is to ensure that the design level of in-elastic response will be provided by a LLRS under earthquake loading.

The upper bound of the displacement window is the maximum code-specified displacement or drift for ULS loading, which in New Zealand is 2.5%. The lower bound of the displacement window is the displacement at which yielding occurs. This depends heavily on the structural system, the structural materials, and the geometry of the structural members and connections. It can be reduced by increasing the stiffness of the lateral load resisting system, but this may be difficult or expensive for some structural timber systems.

4 - PDE AND CPE

Seismic design actions are established in the New Zealand seismic loading standard NZS 1170.5 [11], which defines ***Potential Inelastic Zones*** whose performance must be considered when assigning the structural ductility factor, μ. These zones do not sit well with timber design, as the timber itself is inherently brittle under common loading cases. As such, timber buildings rely on elements, not zones, for their structural ductility, in contrast to reinforced concrete and steel structures.

Similarly, the term inelastic is problematic in timber. For example, a bolted connection which is designed to have an embedment-only failure, failure of either the timber or steel only without fastener yield, will present inelastic response but cannot be considered ductile. Appendix A of NZS 1170.5 requires ductile response to consist of plastic deformations combined with energy dissipation under cyclic loading. For timber, this typically means that the steel fastener will need to yield for a joint to be considered ductile, limiting the failure modes that are possible. As such, the term ductile, rather than inelastic is used to describe ***Potential Ductile Elements*** (PDEs).

For any structure with a selected structural ductility factor μ greater than 1, capacity design must be used. Capacity design in the New Zealand Building Code requires elements of the primary earthquake-resisting system, in this case the PDEs, to be chosen and suitably designed for energy dissipation. To ensure that the PDE is the weakest link, capacity design guarantees that all other structural elements are provided with sufficient strength so the chosen means of energy dissipation can be maintained. These elements are called Capacity Protected Elements, CPEs.

While capacity design is common throughout the world of seismic design its adoption and implementation takes on many forms. While some standards amplify the ductile demand or provide separate demand equations which CPE need to resist, the New Zealand Building Code leaves it to the individual material standard to define what needs to be considered in CPE design. Chapter ZZ9 therefore sets out what effects need to be considered when establishing the required gap between the strength of the PDE and that of the CPE:

- The actual strength of the PDE rather than the calculated strength. This is sometimes referred to as the model bias or model uncertainty and allows for the fact that design equations, that are often based on the interpretation of or validation with testing results, tend towards a conservative representation of strength.
- The 95th-percentile strength of the PDE rather than the characteristic strength. In this context the 'characteristic' strength is intended as the 5th-percentile strength of the connection. Related to the above model bias, however, this also stems from the common use of 5th percentile material strengths (e.g. timber density or steel yield strength) in the design of the connection.
- Material strain hardening. Relevant to the use of steel fasteners, this allows for the fact that the strength of steel increases post-yield,

https://doi.org/10.52202/080513-0405

especially under cyclic loading where the strain plateau is not evident.

- Secondary effects such as friction, rope effect and similar. A general catch all in the standard to state that any other effect that might increase the strength of the PDE needs to be considered.

While the considerations required are stated in the standard, only one actual value for the overstrength is stated with mild steel, screws and dowels being assigned an overstrength factor of 1.6. While it was considered highly desirable that other overstrength values were explicitly stated in the text it was not possible to find general consensus in either industry or research as to other commonly used fasteners and their appropriate overstrength values.

One important point to note is that the standard states that the CPE shall be designed for the 'maximum likely strength of the PDE'. This means that the demand on the CPE is calculated by applying the overstrength to the characteristic capacity, the capacity without a material reduction factor or material partial safety factor and not the PDE demand. This nuance is often missed in design meaning that any 'over design' is not captured in CPE demand. The only exception to this is if the overstrength cut off is used as described in Section 5.3 below. NZS AS 1720.1 states that the material reduction factor can be taken as unity when verifying the adequate strength of the CPE under PDE overstrength demand.

4.1 BRITTLE CONNECTION FAILURE AS A CPE

When designing a PDE that uses dowel type fasteners to provide ductility, NZS AS 1720.1 requires the use of a 'detailed' design method to calculate the strength of the fastener group. This detailed method adopts the European Yield Model in a similar way to Eurocode 5 [2] which differed from the previous design method in New Zealand which provided simplified tables. The use of the European Yield Model allows the designer to ensure that an embedment only failure is not the governing failure mode of the fastener.

NZS AS 1720.l provides two performance points for nails, screws and rivets (classified as small diameter fasteners), the fastener group design yield strength, $N_{y,y}$ and the fastener group design ultimate strength $N_{y,u}$. During the lateral loading of the fastener group, failure or fracture of the timber can occur and will eventually be the failure mode of many joints. This timber fracture or failure is classified as a 'brittle failure modes'. Based on when the brittle failure mode occurs the standard defines the failure of the fastener group itself as brittle, mixed or ductile as shown in Figure 1 below:

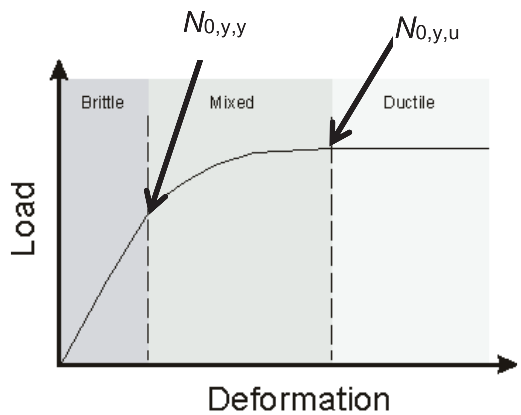

Figure 1. Load deformation curve of fastener group [13].

As shown in Figure 1, the fastener group failure is classified as brittle if the timber fails prior to achieving the fastener group yield strength and mixed if the timber fails prior to reaching the fastener group ultimate strength. NZS AS 1720.1 classifies the 'brittle failure modes' as CPE and they therefore need to be capacity protected. Once a designer has defined their required fastener group by comparing its group design yield strength against the ductile seismic demand, the overstrength factor is applied to the $N_{y,y}$ and the brittle failure modes are checked against this demand. Subsequently to the publication of the standard questions have been raised if the use of $N_{y,y}$ in design is too conservative as it sets the performance point of the joint under seismic loading at a pre-yield state. Considerations of the use of a point between $N_{y,y}$ and $N_{y,u}$ and a subsequent modification of the overstrength factor are suggested.

For bolts, dowels and coach screws, defined as large diameter fasteners, only the design ultimate yielding strength, N_y is calculated. Therefore, this is defined as the performance point of the fastener group and the overstrength factor is applied to N_y for the protection of the brittle failure mode CPE. The fact that the standard defines different performance points for small diameter and large diameter fasteners creates an incongruency between the design of the fastener groups that needs to be addressed with future research.

4.2 SEISMIC PERFORMANCE BEYOND THE ULTIMATE LIMIT STATE

Seismic design in New Zealand explicitly requires two design considerations:

- Buildings shall have a low probability of collapse throughout their lives known as the ultimate limit state (ULS) and;
- Buildings shall have a low probability of loss of amenity throughout their lives known as the serviceability limit state (SLS)

AS/NZS 1170.0 defines for typical buildings with a design working life of 50 years that the annual probability of exceedance for earthquake design becomes 1/500 years and 1/25 years for ULS and SLS, respectively [9]. NZS 1170.5 then defines the horizontal design response spectrum that an engineer can use to calculate the base shear to be applied for the limits states above.

Beyond this explicit expectation however, the commentary to NZS 1170.5 states that:

"It is inherent within this Standard that, in order to ensure an acceptable risk of collapse, there should be a reasonable margin between the performance of material and structural form combinations at the ULS and actions associated with structural failure through a loss of strength or stability. For most ductile materials and structural configurations it has been assumed that a margin of at least 1.5 to 1.8 will be available."

NZS1170.5 then effectively leaves the decision on how this is achieved to the individual material standards meaning this consideration, while not explicitly stated, is expected to be addressed.

The requirement in the NZ standard is similar to the robustness requirement proposed overseas, with concepts still to be fully developed and codified for timber buildings [7] [14].

5 - LOCAL VS GLOBAL DUCTILE DEMAND

The new standard allows any admissible ductile element to be used in the design of a seismic resisting timber system. This is provided that the designer can adequately demonstrate the ability of the element to resist the seismic demand in a stable non-linear manner. Critical to this is the designer's ability to convert the global ductile demand, which determines the Seismic Classification, to a local ductile demand, which is the inelastic demand on the PDE.

Taking a light timber framed shear wall as an example calculation of the local ductile demand is as follows:

1. The global ductile demand is selected, the structural ULS base shear and wall in-elastic demand is calculated
2. The local PDE, typically the nails or screws holding the sheathing to the framing, is selected and the required size and spacing of the fasteners is calculated.
3. The CPEs, typically the studs, sheathing and hold downs are sized for the overstrength demand from the PDE.
4. The elastic deformation of the wall under the ULS demand is calculated, it is then multiplied by the global ductility to find the in-elastic displacement of the wall.
5. As all the CPE have been designed with overstrength the in-elastic deflection will be concentrated in the PDE. The designer can calculate the in-elastic demand on the fastener, by removing all elastic deformation components not related to the PDE. This assumption is slightly conservative as any post-yield increase in fastener strength will also increase the elastic deformation of the CPE.
6. The in-elastic demand on the PDE is then compared to the local deflection and/or strain limits of the PDE.

5.1 LIMITS ON LOCAL DUCTILE DEMAND

The performance based framework of NZS AS 1720.1 requires a specific check of local ductile demand in the verification of the LLRS global ductile response. A limited selection of local deflection and strain limits are provided in the new standard for common fastener types as shown below:

Table 1. Strain or deformation limits for specific PDE

Potential ductile element	Displacement (mm)	Strain in steel (%)
Connections with mild steel nails	6	-
Mild steel rods/bars/bolts – tension only	-	8

https://doi.org/10.52202/080513-0405

Mild steel rods/bars/bolts – compression and bending[1]	-	5
[1] Mild steel rods/bars/bolts that are subjected to compression or bending shall be adequately restrained to prevent local or lateral torsional buckling.		

While Table 1 lists some common PDE, it clearly does not provide the requirements for all PDE possible. In this case the designer must establish their own reasonable limit of deformation or strain capacity. The standard cites 'special study' as being required, a term which has an informative meaning in the New Zealand standards framework where information not provided by the standard is required [9]. The use of ISO 16670 [4] or AS/NZS 1170.0 Appendix B can be considered adequate special study to establish the displacement limit. Key to this however is that a margin must be left between the acceptable ductile limit of the PDE and its ultimate ductile limit (i.e. the point at which the PDE fails). This is to ensure capacity beyond the ULS demand is present in order to satisfy the assumptions within New Zealand's seismic demand framework as described in Section 4.2 above.

While the application of displacement and strain limits is an important verification in the design of a timber structure, judgement should be used in their application. It is often acceptable that a small number of individual fasteners are displaced slightly beyond the limits above provided that the average performance of the PDE is below these limits. For this reason, it is normally appropriate to perform the check on the average ULS displacement/strain of the fastener group and not the absolute maximum value. An example of this is the verification a group of dowels working as a wall hold-down. While the outer-most dowel will be subjected to the highest displacement demand under overturning, the check can be done at the centre of the dowel group.

5.2 FURTHER REQUIREMENTS FOR BRITTLE STRUCTURES

As mentioned above, timber as a material is inherently brittle under its most common load case of tension, compression (buckling) and bending. This creates a potential vulnerability in some timber LLRS when there is no identified PDE. In cases such as this the designer is required to increase the elastic seismic demand by 50% to provide structural resilience. Structures where this increase may apply are pole structures where the pole strength governs resistance (not the embedding soils) or portal frames with curved or glued knees and arch structures where timber strength governs.

It is important to note that not all structures which are designed for elastic seismic loadings are required to be classed as brittle and a degree of engineering judgement should be used. A designer may choose to design a portal frame structure with a bolted gusset or a light timber framed shear wall with nailed plywood sheathing for elastic loading due to another limit state, such as deflection or wind loading governing its design. In these cases they are not required to increase loading by 50% or undertake capacity design. The use of a structural performance factor of 1 in these cases is considered sufficient to provide a margin against collapse although a designer should ensure that disproportionate failure would not occur (i.e. the bolted gusset is not an order of magnitude stronger that the timber members it connects. It is noted that the introduction of a Structural Performance Factor larger than unity for timber structures is conservative, as the seismic loading standard already accounts for the poorer performance of brittle and structure with nomina ductility by specifying higher values of S_p when compared to structures of nominal or full ductility [10]. An additional two storey limitation exists for structures identified as brittle.

5.3 ELASTIC CUT OFF

During the writing of the standard it was noted that the use of lower values of global ductility and the requirement that capacity design is used for all structures with target global ductility beyond elastic, CPE demands may exceed those of an elastically responding structure. This was considered overly onerous in design and an elastic cut off was introduced to limit CPE demand. The cut off limits the CPE demand to that of an elastic structure, but with the allowance for the S_p factor to remain as that of the structure with its chosen target ductility. The overstrength cutoff is therefore de-coupled from the strength of the PDE, and any over-strength or over-capacity as shown below:

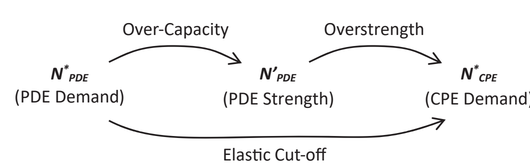

Figure 2. PDE and CPE relationship with over-strength, over-capacity and the elastic cut off [6]

https://doi.org/10.52202/080513-0405

Since the release of the standard this elastic cut off has been re-visited and its appropriateness questioned as for structures with low levels of target global ductility and high levels of over-capacity CPE demand can drop below the PDE strength. Suggestions have been made that the overstrength cut off should set the S_p factor also to 1 for structures with a lower target ductility to avoid this inversion and that in some cases the re-introduction of the material safety factor in the CPE strength calculation may be appropriate. For more information regarding the elastic cut off and the above suggestions please refer to Smith 2023 [6].

6 - KNOWLEDGE GAPS AND AREAS OF RECOMMENDED FUTURE RESEARCH

During the writing of the standard several knowledge gaps were noted where either no information was available or the status of current research was not considered adequately tested or 'standard ready':

- Overstrength (general definition and specific values for bolts and fully threaded screws): While a significant amount of information is available with regards to overstrength a general consensus in its definition is lacking. In addition to this, finite numbers for commonly used systems are lacking that are applicable to real building design.
- High mode effects: As timber buildings get taller the impact of higher modes on seismic response becomes more important. This is particularly true of timber structures where a flexible structural system can mean that second and third mode response becomes dominate, especially in wall or column shear force. NZS AS 1720.1 addresses this by requiring time-history analysis or model response spectrum analysis for buildings beyond four stories however it would be desirable to have methods of calculation to remove this requirement.
- High displacement demand with a pinched hysteretic loop: During the writing of the standard it was noted that forced based design is normally reliant on the assumption that the global ductile response of the system is elasto-plastic with little reduction in hysteretic damping during cyclic response. For many timber LLRS where dowel action is relied upon this is not the case and questions were raised if this needed to be considered in design. For more information on this phenomenon and its potential consideration please refer to Wright (2024) [15].
- Strain and displacement limits: As stated above NZS AS 1720.1 requires a designer verify that PDE local displacement and strain limits remain within their capabilities and with a margin beyond demand to avoid failure under higher than anticipated loading. Well established values as well as consistency of methods in their calculation was found to be scare highlighting an area of further research requirements.
- Definition of consistent capacities for small and large dowel fasteners (yield versus ultimate yield capacities, respectively) and the appropriate level of overstrength to avoid connection brittle failure modes.

7 – CONCLUSIONS AND RECOMMENDATIONS

The NZS AS 1720.1 working group faced an interesting and challenging process in the writing of the new seismic chapter. The idea of performance-based design is deeply ingrained in the seismic design process in New Zealand however globally there is a tendency to be more prescriptive when allowing for system response in seismic design. To ensure performance-based design was still at the heart of the standard clauses the following terms and practices were introduced:

- Clarity around the seismic classification of timber lateral load resisting systems was introduced. This classification is independent of the lateral load resisting system used although some recommendations as to the expected seismic classification is provided. The choice of a seismic classification increases the onus on the structural engineer in design.
- Due to the unique nature of timber where stable levels of ductile response are provided by elements within the system rather than zones. Potential Ductile Elements (PDEs) were defined. Any element that is not potentially ductile is defined as a Capacity Protected Element (CPE). The definition of CPE includes brittle failure modes within the PDE itself.
- New Zealand has two limit states in design; the Ultimate Limit State (ULS) and the

https://doi.org/10.52202/080513-0405

Serviceability Limit State (SLS) however there remains an expectation that a reasonable margin exists between the performance of a material at ULS and loss of strength or stability. For this reason, the standard explicitly requires a designer check the in-elastic demand of a PDE for nominally and limited ductile structures.

Rules were derived to ensure the robustness of systems chosen while allowing greater freedom to designers in their choice of system. Ensuring timber buildings remain a valid response to New Zealand's extreme seismic hazard. However several areas of future research were noted that would greatly enhance the implementation of robust seismic design:

- Overstrength (general definition and specific values for bolts and fully threaded screws)
- High mode effects
- High displacement demand with a pinched hysteretic loop
- Strain and displacement limits
- Consistent definition of fastener performance

8 – REFERENCES

1. CSA Group, *CSA O86:24 - National Standard of Canada: Engineering design in wood*. 2024.
2. European Standard, *EN 1995-1-1:2004+A1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings*. 2004.
3. European Standard, *EN 1998-1 - Eurocode 8 - Design of structures for earthquake resistance*. 2005.
4. ISO, *ISO 16670:2003 - Timber structures — Joints made with mechanical fasteners — Quasi-static reversed-cyclic test method.* 2003.
5. New Zealand Building Code (NZBC). *Building Regulatory Framework*. 2021; Available from: https://www.building.govt.nz/building-code-compliance/how-the-building-code-works#jumpto-building-regulatory-framework.
6. Smith, T., *Timber Unlimited Technical Note - Capacity design and NZS AS 1720.1:2022.* 2023.
7. Sørensen, J.D., et al., *Guideline - Design for Robustness of Timber Structures. COST Action E55 "Modelling of the Performance of Timber Structures".* 2010.
8. Standards Australia, *AS 1720.1:2010 - Timber structures Part 1: Design methods*. 2010.
9. Standards New Zealand, *NZS 1170:0:2002 - Structural design actions - Part 0: General principles*. 2002.
10. Standards New Zealand, *NZS 1170:5 Supp 1:2004 - Structural design actions - Part 5: Earthquake actions - New Zealand Commentary*. 2004.
11. Standards New Zealand, *NZS 1170:5:2004 - Structural design actions - Part 5: Earthquake actions - New Zealand* 2004.
12. Standards New Zealand, *NZS 3603:1993 - Timber structures standard.* 1993.
13. Standards New Zealand, *NZS AS 1720.1:2022 - Timber structures Part 1: Design methods*. 2022.
14. Wood Solutions, *WS TDG 39 Robustness in Structures 07-2023.* 2023.
15. Wright, T., *Seismic performance of high-capacity screwed hold-down connections in mass timber buildings.* . 2024, University of Canterbury.