Study Report

SR400 [2018]

State of the art of timber-based hybrid seismic-resistant structures

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Preface

This is the only output to be prepared within the project QR1630 Hybrid Timber Structures. It presents a review of the available literature on the topic to date and includes descriptions of hybrid timber structural systems and methods for evaluating these systems. These methods including shake table testing, connection testing and numerical analyses of a variety of different hybrid timber structural systems that have been investigated and implemented around the world.

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Abstract

The use of timber in the construction of multi-storey buildings in seismic regions has been gaining momentum around the world for several decades. As timber is a relatively new material for the construction of such structures, engineers face new challenges related to the evaluation of the behaviour of these structures during earthquakes. There has been significant research conducted on structures made of light timber-framed (LTF) walls as well as massive timber such as laminated veneer lumber (LVL) or cross-laminated timber (CLT). However, there are still many questions that remain unsolved. Among those, the commonly accepted practice of relying on the dissipation of the seismic energy to take place in the connections needs to be revisited in the context of new paradigms such as robustness and resilience. To achieve increased performance, the use of timber alongside other materials such as concrete and structural steel has been investigated in the past, leading to the term ‘hybrid structures’. Even though the existence of a minimum amount of hybridity in any building is difficult to deny, a structure where timber is intentionally used in conjunction with structural members of other materials is understood to be a timber-based hybrid structure. This report presents a review of the current body of research on this topic. It covers experimental and numerical research, built applications, connections between elements and design methodologies related to hybrid timber structures. This report discusses the benefits and downsides of the previous research and applications to establish some background for new hybrid timber systems to be proposed and investigated by BRANZ in the future.

Keywords

Hybrid structural system, timber buildings, seismic design.

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1. Introduction and report scope

The use of timber in the construction of multi-storey buildings in seismic regions has significantly increased in recent years. As timber is a relatively new material for the construction of larger structures, engineers face new challenges related to the evaluation of the behaviour of these structures during earthquakes. Despite a range of research data available on structures made of massive timber such as laminated veneer lumber (LVL) or cross-laminated timber (CLT), there are still many questions that remain unanswered, particularly where timber is used in conjunction with other structural materials. Among those, the commonly accepted practice of relying on the dissipation of the seismic energy to take place in the connections needs to be revisited in the context of new paradigms such as robustness and resilience. Robust and resilient structures are those that not only provide life safety but also aim for low damage and rapid reoccupation following a large seismic event.

Innovations in structural systems, improvements in computer analysis and changing legislation around the world have all contributed to a rising interest in larger timber structures. The applications of timber alongside other materials such as concrete and structural steel to achieve improved performance have been investigated in the past, leading to the term 'hybrid structures'. While the existence of a minimum amount of hybridity in any timber building is difficult to deny, a structure where structural timber is intentionally used in conjunction with structural members of other materials can be described as a timber-based hybrid structure. The benefits of these hybrid structures include more efficient use of materials and improved seismic resistance. Increased efficiency can be obtained because the strength, stiffness and weight of particular materials can be utilised where it is most beneficial within a building. Seismic performance can similarly be enhanced by strategically combining materials to optimise ductility, strength and stiffness, all of which contribute to the dynamic structural behaviour of buildings.

This report presents a review of literature on the current knowledge surrounding timber-based hybrid buildings. It covers experimental and numerical research, built applications, connections between elements and design methodologies related to hybrid timber buildings. This report examines the benefits and shortcomings of previous research and applications and identifies where gaps currently exist for the continued development of these buildings.

1.1 Research motivation

The construction of timber multi-storey buildings in seismic regions is not a new concept. It has been on the increase around the world as timber becomes a more popular choice for architects, building owners and developers. Countries such as Japan have a long history of timber buildings that can effectively resist earthquakes. More recently, Canada, New Zealand and Italy have been developing structural systems for larger buildings that can withstand earthquakes with minimal damage, where many of these are hybrid buildings.

The development of robust and resilient structures constitutes a continuing challenge for the structural engineering community as populations increase in urban areas and demands for taller buildings using a variety of materials are on the rise. When these structures are designed for construction in seismically active regions, the complexity of the design significantly increases. To cope with issues that are inherent to primarily timber structures, the use of other materials such as reinforced concrete and structural steel as well as passive control devices including external dissipaters or base isolation have been considered. The success of new structural hybrid structures incorporating timber requires a review of the current knowledge and a discussion on the pros and cons of the most successful investigations to consider when developing designs of these buildings.

1.2 Objectives and scope

The purpose of this investigation is to report on the state of the art of timber-based hybrid structures designed to resist seismic actions. The main objectives are to:

- summarise the types of timber-based hybrid structural systems that have been investigated to date and what methods of verification have been employed
- present examples of timber-based hybrid buildings currently constructed or planned for construction
- identify major code provisions and design methods that impact the implementation of larger timber-based hybrid structures
- identify gaps in current knowledge and make suggestions for future research that would provide critical information on the design and performance of timber-based hybrid buildings.

1.3 Report overview

Section 2 presents a review of experimental testing carried out under quasi-static loading regimes and shake table excitations and including experiments on subassemblies (i.e. portions of a building) and scaled building models. Section 0 covers numerical investigations at component and systems levels, using micro and/or macro modelling of the structural members. Section 4 shows the most relevant aspects related to the connections between the structural elements of different materials. Section 5 includes case studies on built structures with timber-based hybrid systems. Section 6 considers existing design methods, code provisions and guidelines for timber-based hybrid buildings. This includes those methods aimed at determining a reduction factor of an elastic design spectrum in the context of a force-based approach and others that make use of displacement-based design considerations. Finally, section 7 presents a brief summary of the report and makes suggestions on future research in this area.

2. Experimental research

2.1 Introduction

The primary methods for experimental testing of structures subjected to simulated seismic actions are (Dimig et al., 1999):

- quasi-static
- pseudo-dynamic
- shake table tests.

In the quasi-static method, mostly used at a component level, the seismic actions are simulated using actuators that impose a certain displacement or loading protocol upon a specimen. This results in specimens being subjected to slow-motion reversed cycles at increasing displacement amplitudes. The pseudo-dynamic method combines numerical response analysis with experimental testing. Displacements are imposed upon a structural system depending on its restoring forces, measured at every time-step, and the inertial and damping forces simulated on a computer (Thewalt & Mahin, 1987). This method can be applied at different loading rates, although problems can arise with increasing speed because the 'experimental' inertial and damping forces become non-negligible and are already accounted for in the testing algorithm (Dimig et al., 1999). Shake table tests represent the most realistic method for simulating the response of structures against earthquakes. The test imposes the motion of the ground, in one or more directions, to the base of a specimen to obtain its complete response in real time, without the need of simulations and a priori assumptions about inertial and damping forces.

Other experimental methods have also been reported in the literature. These include:

- displacement-controlled dynamic testing
- effective force testing method
- real-time dynamic hybrid testing methods.

All these techniques can be classified as variations of the pseudo-dynamic method. Displacement-controlled dynamic testing is mostly applicable at a component level and involves the application of a displacement history that emulates the response of a part of a structure, as obtained with computer simulations, or as the recorded displacement of the ground. The effective force testing method, discussed in detail in Dimig et al. (1999), consists of the application of an effective force F_{eff} = $-mx_g(t)$, where m is the mass of a single degree of freedom (SDOF) structure, for example, and $x_g(t)$ is the ground acceleration history. Real-time dynamic hybrid testing methods combine shake tables, actuators and computational engines such that either the force or the displacement of a structure is controlled but not both. This technique can include a shake table at the base of a specimen and an actuator at the top such that the boundary conditions are imposed in both levels (Reinhorn et al., 2004).

This section summarises the experimental research on timber-based hybrid structures carried out to date. These investigations included quasi-static, pseudo-dynamic and shake table tests at component and system levels, as described in the following sections.

2.2 Quasi-static and pseudo-dynamic tests

He and Li (2012) and He et al. (2014) proposed a hybrid system consisting of a steel moment-resisting frame infilled with light timber-framed (LTF) wall panels. They tested two full-scale single-storey, two-bay assemblies with three hybrid frames each. These specimens had double and single-sided oriented strand board (OSB) panel sheathing. An example of a single-sided specimen is shown in Figure 1(b).

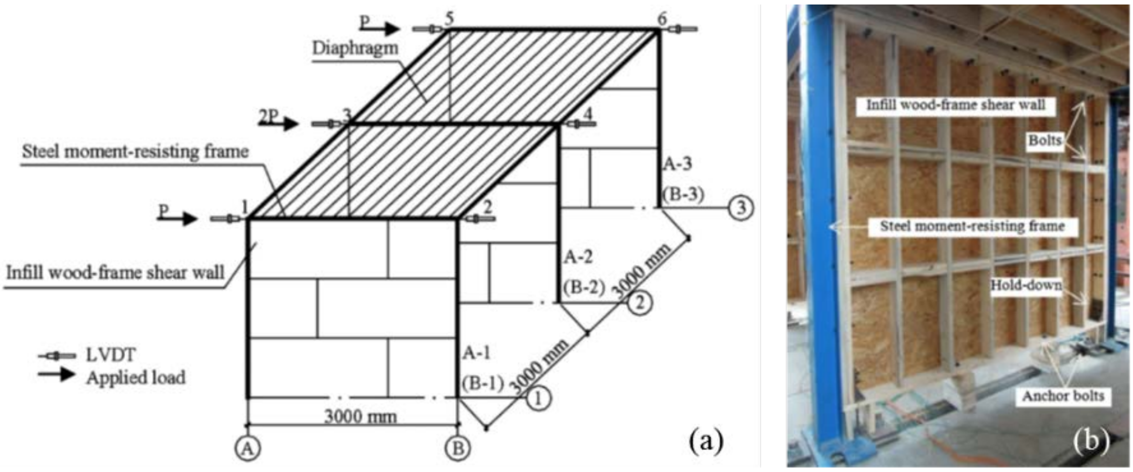


Figure 1. (a) Specimen dimensions; (b) single hybrid frame type A.

(Reproduced from He et al., 2014, with permission from ASCE.)

The hybrid specimens were tested using quasi-static reversed cyclic loading with increasing amplitude. Additionally, two bare steel frames were tested to understand how the inclusion of the timber wall influenced the response of the hybrid system. The authors found the inclusion of the infill timber wall resulted in a substantial increase in the initial lateral stiffness when compared to the bare steel frame. Specimen B had approximately twice the initial stiffness of specimen A, as well as greater strength at the yielding point. The hysteresis plots of the hybrid system showed a pronounced pinching effect but much less noticeable than in the case of the timber wall alone. It was also found that the total imposed lateral load was resisted almost entirely by the timber infill at low displacement levels. After the infill walls were damaged, their effective resistance to load was reduced significantly and the steel frame began to take more load. This effect was more pronounced for specimen A than for specimen B.

As per benefits of this system, the use of a steel frame outside the timber wall significantly improved the hysteresis behaviour of the latter, reducing the pinching and making the system more ductile. However, the behaviour of the steel frame was not enhanced by the timber-infill, leading to the question of how this system was beneficial over simply using the steel frame.

Zhou et al. (2014a) proposed the use of reinforced concrete masonry (RCM) walls within multi-storey LTF buildings to increase the lateral stiffness of the system, thereby reducing the expected displacements resulting from seismic actions. The authors carried out an experimental investigation that included one single-storey LTF wall, one 2-storey RCM wall and two 2-storey LTF-RCM hybrid walls (specimens HW1 and HW2 as shown in Figure 2). The specimens were tested under a reversed cyclic quasi-static loading protocol using increasing displacements. The force was applied at the top of the hybrid specimens and was fixed to the LTF and RCM members for the tests of specimens HW1 and HW2, respectively.

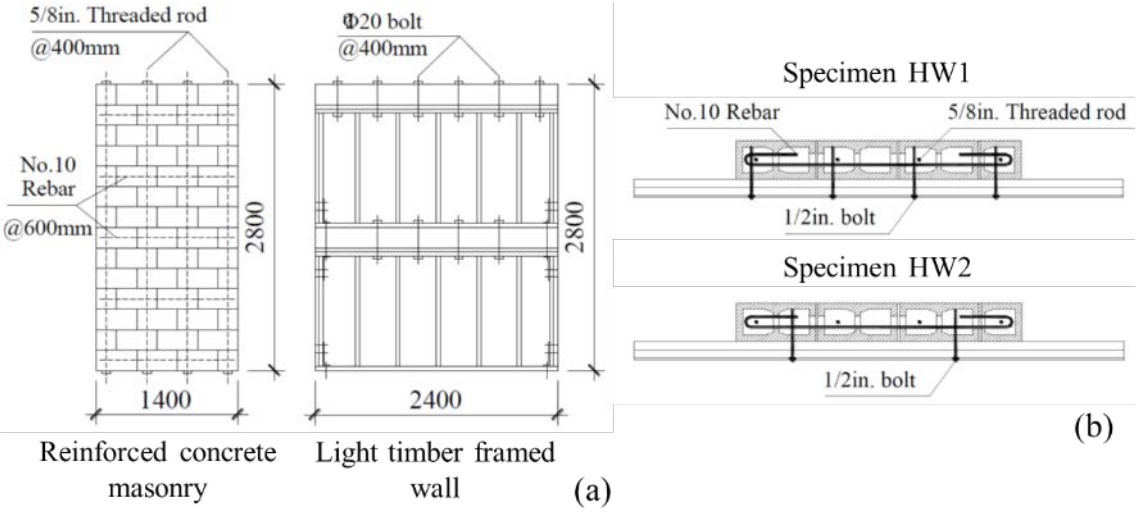


Figure 2. (a) RCM and LTF walls specimen elevations; (b) plan views of hybrid specimens.

(Reproduced from Zhou et al., 2014a, with permission from ASCE.)

As reported by Zhou et al. (2014a), during the test of HW1, the connection between the LTF and the RCM walls failed first. This was a consequence of the lateral strength of the RCM wall being larger than the shear capacity of the anchor bolts. During the test of HW2, the RCM wall failed first due to the larger displacements based on failure of the LTF when compared to the RCM wall and the capability of the connections to transfer the load resisted by the LTF to the masonry. The results showed that the hysteretic behaviour of both hybrid specimens was dominated by the behaviour of the RCM wall, characterised by low energy dissipation capacity and marked pinching, in this case, without strength degradation.

Despite the intention of this research to use a stiff structural member such as RCM walls to control the displacements that a flexible system as a tall LTF building could face during seismic events, there were aspects that deserve attention. There were significant differences in the ultimate displacement capacity, strength and stiffness of RCM and LTF walls. As the RCM wall was much stronger and stiffer than the LTF wall, it carried most of the inertial forces transmitted to both resisting elements. This is the opposite of what occurred in LTF-infilled steel frames (He & Li, 2012; He et al., 2014). Additionally, as the ultimate displacement of the masonry wall is smaller than that of the LTF wall, the behaviour of the hybrid system will be controlled largely by the former, leaving the latter ineffective for all practical purposes.

The methodology followed in the experiment also presented some problems. As discussed by the authors (Zhou et al., 2014a), the lateral load was applied either on the RCM or the LTF walls, which was not representative of what would occur if these loads were meant to represent the equivalent inertial forces transmitted through a floor diaphragm to the structural elements (Crisafulli et al., 2005).

Pozza et al. (2016) proposed a system formed by platform-frame timber walls (PFTW) with external reinforced concrete (RC) panels screwed to one of its faces, as shown in Figure 3. The RC panels were intended to provide additional bracing to the system therefore increasing the lateral stiffness of the walls.

As shown in Figure 3, the system was modular. Each individual panel or basic modular panel was formed by three square panels, resulting in a vertical-to-horizontal aspect ratio (AR) of 3. Three wall configurations, named A, B, and C, with AR equal to 3, 3/2 and 1, respectively, were tested. In addition, wall C had an opening in the central part (see Figure 3(d)). The walls were subjected to a cyclic quasi-static loading protocol with increasing amplitudes. The load was applied at the top of specimens. The authors discuss the results of tests of walls B and C only, referring the reader to Pozza (2013) for the testing results of wall A.

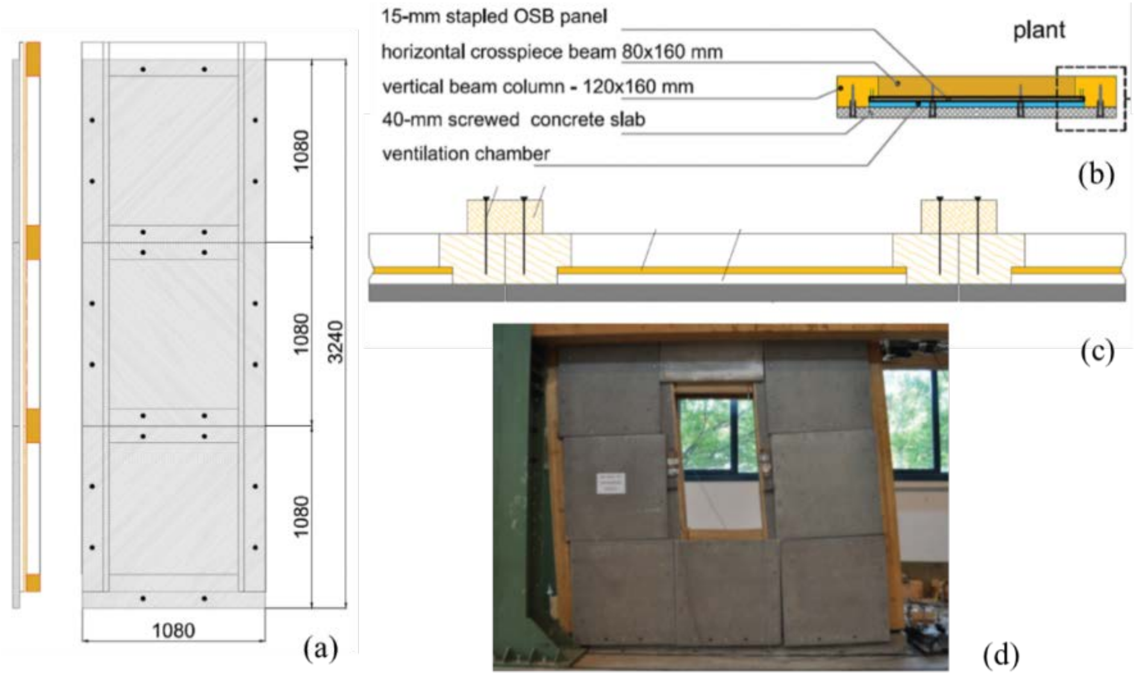


Figure 3. Timber walls plus RC panels: (a) panel elevation; (b) panel plan view; (c) panel side view; (d) specimen C.

(Reproduced from Pozza et al., 2016, with permission from ASCE.)

The experimental results showed that the hysteretic behaviour of walls B and C was characterised by large pinching and strength hardening effects, reflecting the typical hysteresis loops of the connections used in timber construction. The walls showed significant shear deformations because of their squat geometry. The main source of damage in walls B and C at the end of the tests was permanent dislocation of the concrete slabs and inelastic deformations of the anchorage system between the concrete and the timber.

The benefit of this system is difficult to identify, as it did not improve the pinched hysteretic behaviour typical of timber walls. There were also several disadvantages. The most evident was that the connections between the timber and the concrete were the weakest link of the system as they experienced inelastic deformations. This is thought to be one of the main drawbacks of timber construction in seismic regions, and it should be overcome with the addition of other structural materials in the context of hybrid structures. If the connections suffered inelastic deformations, they were more likely to fail, increasing the likelihood of the concrete panels to become detached from the timber. Additionally, it is clear from Figure 3(d) that the tested concrete panels suffered dislocation and did not act monolithically as part of the rest of the structure to effectively increase the stiffness and the strength of the system.

Gilbert and Erochko (2016) proposed to include steel braces within massive timber frames. The intention was to justify the use of larger reduction factors of the elastic design spectrum[1] to reduce the design seismic forces and to improve the seismic performance by increasing the seismic energy dissipation capability of the structure. To investigate the cyclic behaviour of the proposed system, the authors performed quasi-static and pseudo-dynamic unidirectional tests of a half-scale glulam frame assembly that included steel braces with frictional dissipaters and heavy glued-in steel rod connections.

With the quasi-static tests, it was found that inelastic deformations occurred in the friction device only, while the other structural members and connections remained within the elastic range. This was reflected in the force-displacement hysteresis plots, which indicated good capacity design of the system such that this device was the weakest link in the chain. The pseudo-dynamic tests supported this finding. During these experiments, the system dissipated most of the hysteretic energy in the frictional brace, this time accommodating the non-periodic earthquake-induced displacements imposed upon the subassembly.[2]

This investigation showed that, in principle, steel friction dissipating braces can be successfully included in massive timber (glulam) frame structures. As these braces were designed to be the weakest link of the system, following capacity design principles (Park & Paulay, 1975; Paulay & Priestley, 1992), the frictional device protected the timber elements and their connections against suffering inelastic strains that might compromise the integrity of the structure. Another improvement that steel braces can provide to timber-only structures, not discussed by Gilbert and Erochko (2016), is an increase in the stiffness of the system, decreasing its maximum expected deflections during earthquakes. Nevertheless, these findings need to be examined in depth at a system level and in the dynamic rage using shake table tests.

The drawbacks of the system proposed by Gilbert and Erochko (2016) include:

- excessive heating of the frictional device, the result of high velocities during a seismic event
- the potential existence of debris in the friction surface due to local melting of the softer frictional material
- creep of the soft material due to changes in its crystalline structure
- durability aspects mainly related to corrosion (Christopoulos & Filiatrault, 2006; Canterbury Earthquake Royal Commission, 2012).

Blomgren et al. (2016) also proposed to include braces within glulam frames. They introduced a novel timber-based buckling restrained brace (TBRB). Readers can refer to Uang and Nakashima (2004) and Black et al. (2004) for buckling restrained braces used in steel structures. The TBRB included three parts:

- A steel core able to sustain tension and compression strains without buckling.
- A split glulam casing providing continuous lateral restraint to the steel core.
- A de-bonded interface between timber and steel, which in this case was simply air.

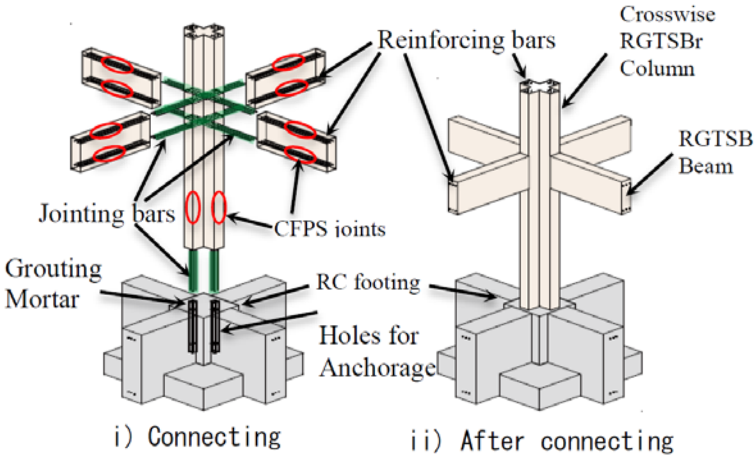
Blomgren et al. (2016) carried out a proof-of-concept test for the TBRB using three full-scale specimens within a glulam frame. The subassembly was tested under unidirectional cyclic quasi-static loading following the requirements of the US standard ANSI/AISC 341-10 *Seismic provisions for structural steel*. The results showed that the

[1] For a discussion on the role of justification on empirical science, see Miller (2011).
[2] Note that Gilbert and Erochko (2016) call these tests “dynamic”, but as the displacement was being imposed during the test, it was in fact a pseudo-dynamic testing procedure.

first TBRB did not meet the acceptance criteria required by this standard due to a higher-mode buckling of the steel core. The second one, on the other hand, did meet the criteria as it could achieve the ductile behaviour required by ANSI/AISC 341-10, presenting stable, full hysteresis loops with large energy dissipation capability.

The work conducted by Blomgren et al. (2016) was a positive preliminary step to consider this type of brace in the construction of multi-storey timber buildings, and the proposed brace was novel. However, aspects related to the robustness of the system remain unaddressed. In theory, it should be proved that the use of timber for restraining the steel core is a better option than using a steel tube filled with concrete mortar, as the robustness of that system is very high and it is easier to fail a glulam sleeve than a steel tube. Also, the former material is brittle and the latter ductile. There is still no experimental information about the behaviour of the TBRB in the dynamic range within a multi-storey model building, and it may not be sufficient to do that analytically as the authors did.

With the aim of upgrading the flexural strength and stiffness of timber frame structures, Shioya et al. (2016) proposed a hybrid system named the reinforced glulam structure system using steel bars (RGTSB) made of glulam timber reinforced with deformed steel bars. These bars were glued to the timber using liquid epoxy resin adhesive and spliced with carbon fibre plastic sleeves. Figure 4 shows construction details of the system.



(b)

Figure 4. (a) RGTSB system details; (b) application of adhesive between steel and timber.

(Reproduced from Shioya et al., 2016, with permission from the authors.)

As shown in Figure 4, the timber members were manufactured with grooves to place the steel bars. After the bars were located, the grooves were filled with epoxy adhesive, forming a monolithic composite member. Shioya et al. (2016) tested one two-thirds scale column and two half-scale beam assemblies. The column was tested under cyclic quasi-static lateral loading with increasing amplitude and included constant axial load applied at the top. The loading protocol included low amplitude cycles, resembling what would be a free vibration portion of a dynamic response. The obtained hysteresis loops reflected stable behaviour with large energy dissipation capability and reduced pinching. Damage in the system was observed at about 2.4% drift ratio. The beams were tested with the same experimental method as the columns, but in a set-up especially designed for imposing large rotations at their ends while deforming in double curvature due to the action of a vertical force. The results showed a similar behaviour to that obtained in the column tests.

The system proposed by Shioya et al. (2016) could have some beneficial applications as it resembled the benefits of reinforced concrete for timber elements. However, there are some concerns about the practicality of the system – for example:

- machining the timber pieces with specific grooves is very labour intensive
- the load transfer mechanism between the steel, epoxy and glulam should be studied in deeper detail, particularly addressing durability issues.

In New Zealand, it is required that the service life of a building be equal to at least 50 years. As such, the efficiency of the system after extended periods of time should be also investigated before it can be confidently used.

2.3 Shake table tests

van de Lindt et al. (2011) reported the results of triaxial shake table tests of a 7-storey full-scale building tested at the E-Defence facilities in Miki, Japan. The building was constructed as a hybrid structure with the ground and upper floors comprised of steel and LTF, respectively, as seen in Figure 5.

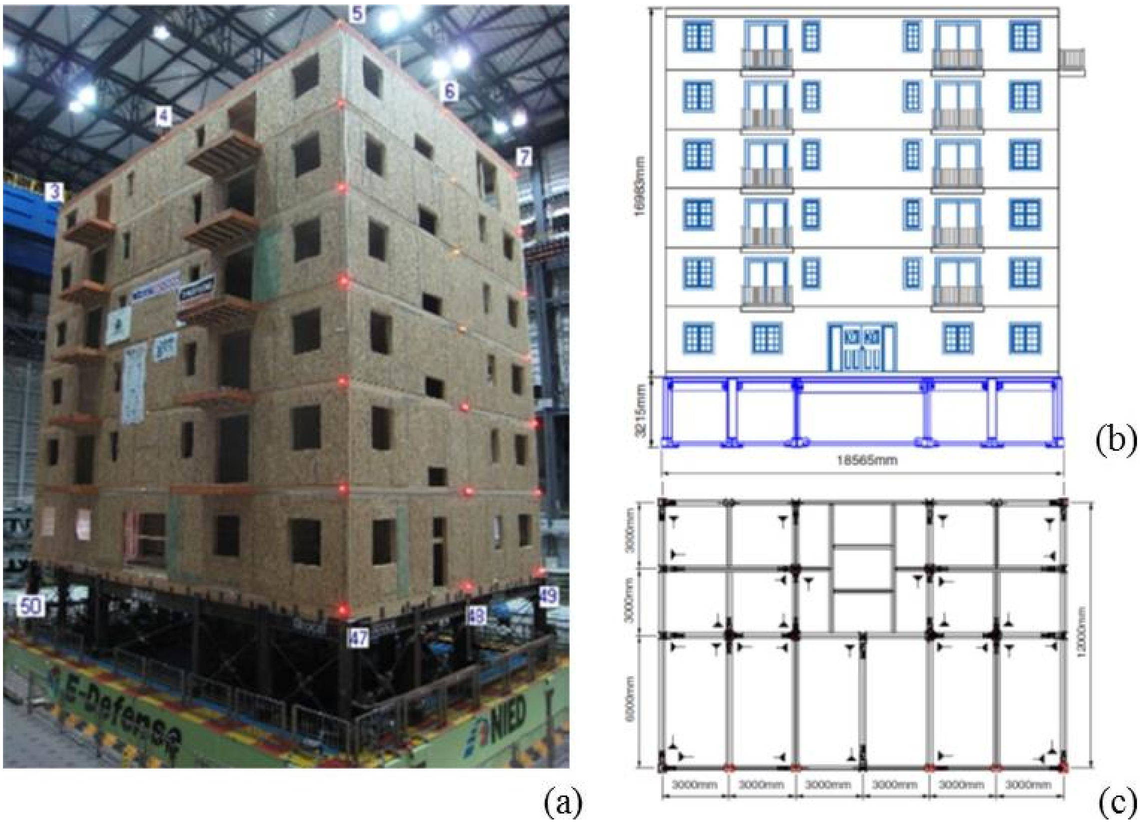


Figure 5. Full-scale 7-storey shake table testing: (a) Picture taken from the north-west (orientation defined in (c)); (b) north elevation; (c) ground floor plan view.

(Reprinted from van de Lindt et al., 2011, copyright (2011), with permission from Elsevier.)

The specimen was tested in the hybrid configuration first (van de Lindt et al., 2011) and then as a timber-only structure (van de Lindt et al., 2012) after stiffening the steel ground floor, converting it in a rigid extension of the shake table surface (phases I and

II, respectively). These experiments provided a comparison between the hybrid and timber-only specimens. However, despite the ground floor stiffening, there will always be some degree of flexibility in that floor when using steel. Additionally, both buildings had a different number of storeys, which made equivalent comparisons troublesome.

The hybrid specimen, addressed in this report, was tested using a ground motion recorded during the 1994 Northridge earthquake at Canoga Park station. The motion, decomposed into the three orthogonal components (two horizontals, X and Y, and vertical, Z), was scaled in amplitude such that, during the first test, it had peak values of 0.19 g, 0.22 g, and 0.26 g in the X, Y, and Z directions, respectively. During the second test, the input motion had peak values (PGA) of 0.50 g, 0.58 g and 0.69 g in the X, Y, and Z directions, respectively, as defined in Figure 5(c). Hence, as is commonly done in shake table experiments, the first and second input motions were aimed at representing a common and a rare event, respectively.

The results showed that, during the first test, the building experienced maximum roof drift ratios (RDR) of 0.05% and 0.32% in the X and Y directions, respectively, while during the second test, the maximum measured RDR reached 0.55% and 0.82% in the X and Y directions, respectively. The measured inter-storey drift ratios (*dr*) reached a maximum in the 5th storey during the second test, with 0.92% and 1.31% in the X and Y directions, respectively. The measured base shear was 60% and 70% of the shear capacity of the steel floor, approximately, which is about 46% of the total seismic weight of the building. van de Lindt et al. (2011) reported minimal damage during the two shake table tests, which was consistent with the small magnitude of the roof and inter-storey drift ratios.

The hybrid construction system that van de Lindt et al. (2011) proposed had a practical advantage in that it enabled the possibility of constructing a building with a timber super-structure that exceeded the restriction related to the maxim number of storeys of these buildings in seismic regions in some parts of the world. However, other benefits such as forcing the damage to occur outside the connections of LTF walls was not addressed. The conclusions reached by the authors related to the performance of the new system may lead to unconservative confidence on the grounds of one series of experiments. It is true that the maximum values of the inter-storey drifts recorded during the building response were rather low. Nevertheless, the fact that such low displacements were measured during the tests does not imply that this will be the case if other input motions are used, including those with smaller PGA, longer duration and ‘peculiar’ frequency content, for example (Quintana Gallo et al., 2017). As shown with the shake table tests carried out by Quintana Gallo (2014), large PGA levels do not necessarily correlate with an intense seismic event that results in extensive damage in traditional structures. Hence, more experiments are required to test the system and draw more robust conclusions.

Jamil et al. (2015) proposed a hybrid system formed by continuous steel columns with corbels, LVL beams and friction dissipating devices to form a sliding hinge joint (SHJ) (Clifton, 2005). The system and connection are illustrated in Figure 6. The SHJ (Figure 6(b)) designed for this case worked as follows: as the column leaned towards the right-hand side, the friction device on the left corbel slid up, resisting additional vertical load, which in turn provided a restoring moment with re-centring action – see Jamil et al. (2015) for details.

Jamil et al. (2015) constructed a one-fifth scale model building with the proposed hybrid system and tested it using quasi-static cyclic loading and unidirectional shake table excitations. The experimental model, shown in Figure 6(a), had one span in the loading and orthogonal directions. In the latter, LTF walls were also included.

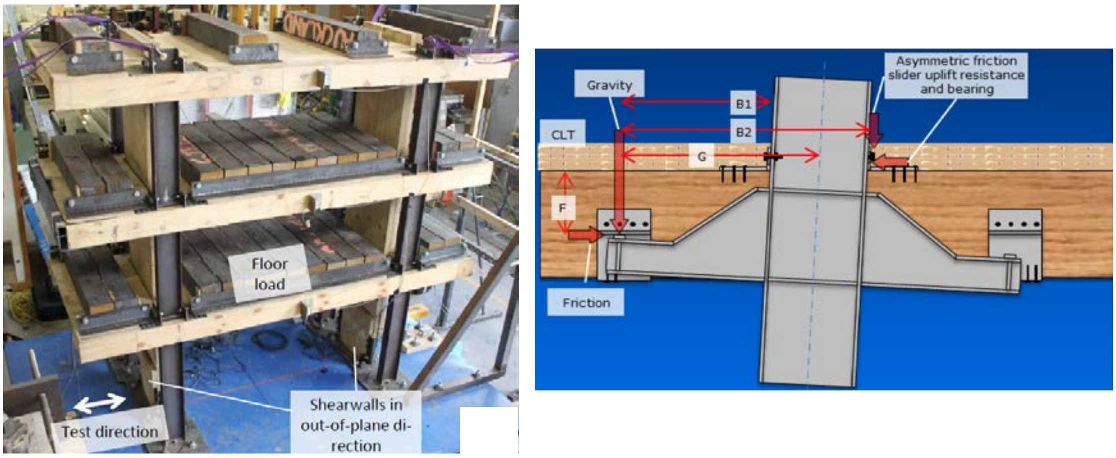


Figure 6. SHJ shake table testing: (a) picture of the building model; (b) detail of the connection.

(Reproduced from Jamil et al., 2015, with permission from NZSEE.)

The results of the quasi-static tests showed that the model remained in the linear elastic range up to 0.5% RDR, followed by a sudden reduction of the stiffness due to opening of the joints. After this nominal yielding point, the stiffness of the system remained increasing at an exponential rate, as reflected in the hysteresis loops presented in Jamil et al. (2015). The authors attributed this increase to the development of large moments at the base of the columns, which were designed to be pinned. The hysteresis loops reflected stable behaviour with no strength degradation up to 4.5% RDR and re-centring capability. The shape of the hysteresis cycles resembled the shape of a flag, similar to that for precast elements with hybrid connections (Priestley & Tao, 1993; Priestley et al., 1999; Priestley, 2003). The authors indicated that the bolts of the top friction slider were not tightened prior to testing.

During the shake table tests, the specimen was subjected to a set of seven records, representing a default suite for the North Island of New Zealand (Oyarzo-Vera et al., 2012), as the prototype structure was assumed to be constructed in Wellington. As required in dynamic experimental testing, the rules of similitude must be consistently used and carefully selected. In this case, the authors selected a replica where the mass of the model was artificially increased to maintain consistency with the use of prototype materials and gravity loads were considered. Consequently, the time must be scaled down by t_r, the square root of the length scaling factor, l_r. In this case l_r = 1/5 = 0.20, hence t_r = 0.45. Thus, the duration of the input motion was approximately half of that recorded in reality. The records were further modified by reducing their velocity to remain below the limit of the shake table. The authors reported that some of the motions could not be imposed due to in situ problems with the velocity. However, they reported the maximum values of the RDR on each test, with an overall maximum close to 4%. The only recorded response shown in the paper shows limited RDR. No significant damage was reported during the tests except for loss of tension in the bolts.

As mentioned before, the use of friction devices presents durability concerns. In addition, the system presented by Jamil et al. (2015) would require further investigation using larger scale models because the behaviour of the friction devices could be very different due to size effects.

Kohara et al. (2016) proposed the use of viscous-elastic dissipating braces inside LTF walls. The braces were connected to the timber elements with relatively light steel plates anchored to the corner of the frame with screws. The braces were installed in the LTF of the full-scale experimental model. The hybrid specimen and another without the braces were subjected to shake table excitation using several ground motions recorded in Japan, including the 2011 Great East Japan earthquake. The experiments showed that important reductions of the roof displacement (or drift ratio) were achieved during the test of the building with the braced LTF. It is important to note that the reduction of displacements could be the result of the increased stiffness of the braced structure and did not necessarily mean that robust energy dissipation sources were triggered.

The system presented by Kohara et al. (2016) constituted a relatively simple solution for upgrading the behaviour of LTF walls. Using viscous dissipaters, the hybrid system provided restoring forces as a function of the velocity of its motion during seismic events. As shown in Christchurch during the 2011 Canterbury earthquake, a near-field event can impose severe damage as the result of high-velocity pulses. Hence, this idea can be of interest for new constructions in Wellington, for example, where the known crust faults are very close to the city. However, a more comprehensive investigation on the system should be conducted prior to adopting it in practice, including highly demanding near-field events and triaxial shaking tests.

Sakata et al. (2016) proposed a similar system to that proposed by Kohara et al. (2016) using passive control techniques conceived for multi-storey buildings applied to LTF construction (Figure 7). This was a modified version of the added damping and stiffness device (ADAS) dissipating system initially proposed by Whittaker et al. (1991) and also recently investigated by Di Cesare et al. (2012). The difference between both systems lies in the location of the damping device, which was connected to the mid-height of a column and the mid-span of a beam in the case of Sakata et al. (2016) and Whittaker et al. (1991), respectively. The dampers were used within K and inverted V (Chevron) braces in the former and latter cases, respectively.

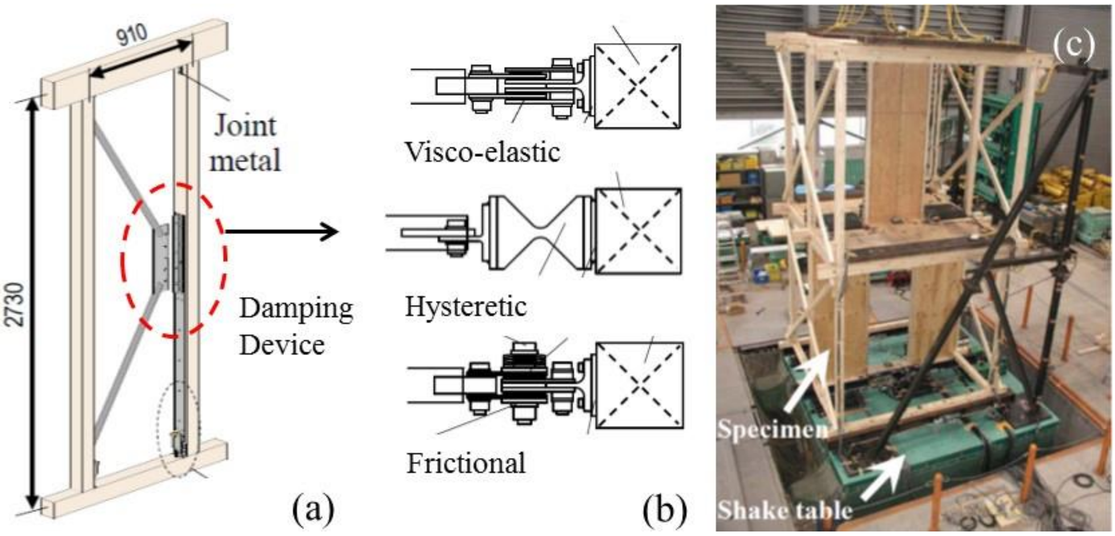


Figure 7. (a) LTF with damping device and braces; (b) types of damping devices; (c) specimen tested on the shake table.

(Reproduced from Sakata et al., 2016, with permission from the authors.)

Sakata et al. (2016) performed displacement-controlled dynamic tests on braced LTF walls with viscoelastic, hysteretic and friction damping devices. The results showed that, in the three cases, the hysteresis behaviour of the unbraced original LTF panel was significantly improved, eradicating the pinching effect and increasing the energy dissipation capability of the system.

Following these tests, Sakata et al. (2016) carried out full-scale unidirectional shake table tests of a 2-storey structure with the three types of braced LTF sheathed walls with none, one and two plywood panels. Benchmark specimens with traditional LTF walls only were also tested. A representative experimental model is shown in Figure 7(c). The specimens were subjected to earthquake motions recorded during the 1995 Kobe (JMA station) and the 1952 Kern Country (Taft station) earthquakes. These ground motions were scaled in amplitude to achieve several peak accelerations (PGA). The results showed that the use of the steel braces with dissipating devices improved the energy dissipation of the system and reduced the maximum displacements of the structure. It was found that, when measuring the fundamental period of the structure before and after the tests, the period of the benchmark specimen increased whereas the period of the braced specimens did not. This indicated degradation of the stiffness of the system (as the mass remains the same) and hence an increase in the cumulated damage. This was consistent with damage observations after the tests.

The system proposed by Sakata et al. (2016) followed similar principles as that proposed by Kohara et al. (2016). As such, the comments on the latter system apply in this case. It is worth mentioning that the viscoelastic dissipaters have an advantage over the hysteretic and friction dampers as they do not accumulate inelastic deformations nor have durability concerns, respectively.

2.4 Summary

Numerous timber-based hybrid systems have been experimentally investigated in recent years. These include light-timber framed (LTF) walls, glulam and LVL frame members combined with steel frame members, RC masonry walls, dissipating devices and braces, as summarised in Table 1.

Table 1. Summary of experimental research on timber-based hybrid structures.

Authors	Timber	Other system	Test
He et al., 2014	LTF walls	Steel frames	Quasi-static
Zhou et al., 2014a	LTF walls	RC masonry	Quasi-static
Pozza et al., 2016	LTF walls	RC panels	Quasi-static
Gilbert & Erochko, 2016	Glulam frames	Steel friction braces	Quasi-static, pseudo-dynamic
Blomgren et al., 2016	Glulam frames	Timber buckling restrained braces	Quasi-static
Shioya et al., 2016	Glulam frames	Reinforcing steel rebar	Quasi-static
van de Lindt et al., 2011	LTF walls	Steel podium	Shake table
Jamil et al., 2015	LVL beams	Steel columns and friction devices	Shake table
Kohara et al., 2016	LTF walls	Viscoelastic braces	Shake table
Sakata et al., 2016	LTF walls	ADAS dissipaters	Shake table

Most of these experiments were quasi-static tests, although one included a pseudo-dynamic procedure, and some others included shake table testing. These tests were performed on subassemblies or structural components in most of the cases. Other tests were conducted using reduced-scale model structures, and one included experiments on a full-scale building.

In most cases the investigated timber-based hybrid systems exhibited improved structural performance over timber-only solutions. In other systems, the benefits were not as clear – for example, when relatively brittle materials were used in conjunction with timber or when the behaviour of the hybrid system was not superior to the non-timber system alone.

Overall, the findings indicated that the more successful hybrid systems included sources of seismic energy dissipation that were separate from the connections, as is traditionally accepted in timber construction. However, many of the systems studied at a component level and/or using quasi-static testing procedures should be evaluated using more realistic methods, ideally including triaxial shake table tests.

3. Numerical investigations

3.1 Introduction

Numerical methods for the seismic analysis of structures can be divided into two main categories:

- Linear elastic analysis, which includes:
  - elastic equivalent lateral load
  - elastic response spectrum modal
  - elastic dynamic analysis.
- Inelastic or non-linear analysis, which includes:
  - non-linear static (pushover)
  - non-linear dynamic analysis (NLDA) (Filippou & Fenves, 2004).

Elastic analyses lack an acceptable degree of realism to estimate the response of structures against seismic actions as they cannot represent the differences resulting from structural systems with different hysteretic characteristics (Priestley et al., 2007). Pushover analyses, most suitable for structures that will predominantly respond in the first mode of vibration, can provide important insight into the relationship between global and local inelastic deformations. However, these deformation estimates can be very inaccurate for structures whose response is strongly influenced by higher modes and may not expose weaknesses that can only become apparent when the dynamic characteristics of the structure change after the first local mechanism forms (Krawinkler & Seneviratna, 1998). NLDA provides the most accurate method of those mentioned above, as it can account for the hysteresis properties of the structural members and include the effects of higher modes of vibration (Priestley et al., 2007). It must be recognised that several modelling assumptions must be made when constructing a model suitable for this method, which sometimes leads to inaccurate predictions (Quintana Gallo, 2014).

Using non-linear analyses, one can choose distinct types of elements, which include line, fibre and three-dimensional solid elements, in order of increasing complexity. Line elements are normally formed by one or more elastic segments and a discrete number of springs where the inelastic behaviour is concentrated (Carr, 2016a). The inelastic properties of these elements are defined with macroscopic hysteretic properties, such as a bilinear or Takeda rules, for example (Carr, 2016b), and require moment-curvature analyses or calibration with experiments. Fibre elements are based on the representation of the cross-section of structural members by a series of fibres, individually representing the properties of a given material.[3] Finally, solid elements are three-dimensional finite elements that can model structures without any a priori geometric simplification. They require significant computational effort and are intended for highly specialised and detailed analyses of complex structures. In the following sections, the numerical evaluation of several timber-based hybrid structures that include predominantly timber are reviewed. They mostly consist of NLDA and pushover analyses of models constructed with line elements having macro-properties. The investigations are divided into analyses performed at a system or a component level, i.e. an entire structure constructed with the hybrid system or the isolated hybrid members, respectively.

[3] For example, in an RC member, some fibres will represent the concrete and others the reinforcing steel bars.

3.2 Analyses at a system level

Asiz et al. (2011) studied the use of steel portal frames within multi-storey LTF buildings to provide larger open spaces in taller buildings. The authors carried out a numerical investigation of the seismic response of hybrid timber-steel and LTF wall timber-only buildings 3 and 6 storeys high.

The building configurations analysed were:

- benchmark structure with standard LTF walls
- a mix of portal frames and standard LTF walls
- portal frames only.

The typical structural plans are shown in Figure 8.

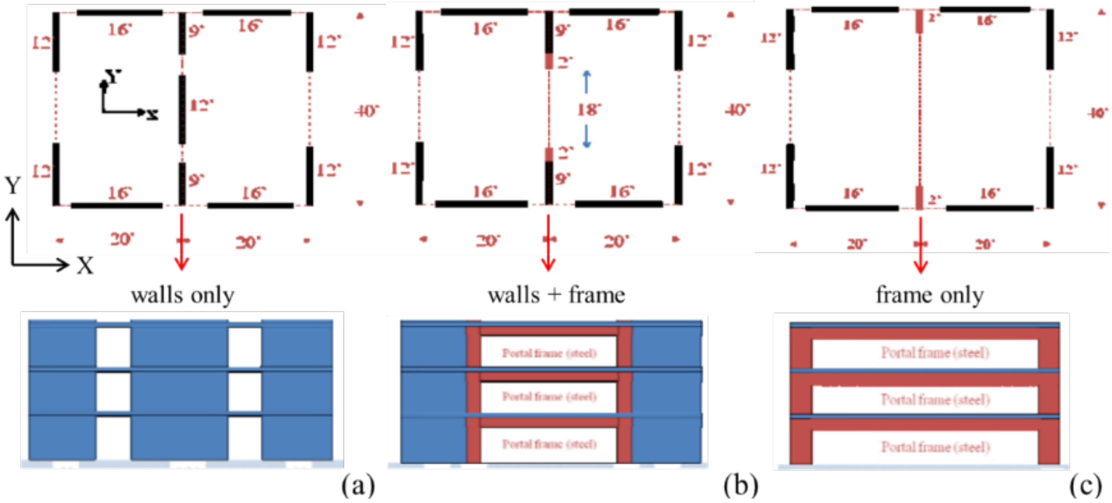


Figure 8. LTF walls and steel frames; central elevation with (a) walls only; (b) walls and steel frame; (c) steel frame only.

(Reproduced from Asiz et al., 2011, with permission from the authors.)

Simplified planar models of the buildings loaded in the Y direction were constructed in the computer program SAPWood (Pei & van de Lindt, 2007). The hysteresis behaviour of the LTF walls and steel members was modelled with macro-elements with a pinching-stiffness degrading rule similar to that proposed by Stewart (1987) and Folz and Filiatrault (2004) and a bilinear rule (Caughey, 1960). To provide an estimation of the inelastic behaviour of the structures, a series of pushover analyses was conducted. With those analyses, it was found that the steel frames did not significantly increase the strength nor ductility of the system. The results showed that the 6-storey structure could be much more brittle than the 3-storey counterpart. The only advantage in the use of steel frames in this case was an increase in the post-yielding stiffness of the system.

Following the pushover analyses, Asiz et al. (2011) performed a series of non-linear dynamic analyses (NLDA) of the structures, using “nine different earthquake records”.[4] These records were scaled in amplitude to simulate different intensity scenarios. The maximum PGA ranged from 0.12 g to 0.56 g. The authors used the information obtained using NLDA to evaluate the PGA associated with the collapse of the model (PGA_c) and the collapse margin ratio (CMR). The former was defined as the PGA at 7%

[4] The authors do not provide any specific information about the input motions used.

drift ratio and the latter as the ratio between PGA_c and the design PGA (PGA_d, equal to 0.46 g in this case). The results showed that the CMR of the hybrid-steel structure was larger than the CMR of the all-timber benchmark. The authors concluded that, in general, it was feasible to mix steel portal frames with LTF walls in multi-storey buildings, but they also acknowledged that laboratory experiments and more detailed numerical models needed to be conducted to study the system in more depth.

The numerical approach used in the work of Asiz et al. (2011) can only provide a coarse estimation of the seismic response of the hybrid system, due to the limited nature of the selected type of analysis (see section 3.1). However, in this case, the pushover analyses provided some interesting findings on the ductility of the system (see Krawinkler & Seneviratna, 1998), assuming that the analysed structure would behave predominantly in the first mode. Replacing the LTF walls with steel portal frames did not increase the ductility of the system, raising questions around the relevance of the fragility curves presented by the authors. It could be the case that the data points used to construct the PGA vs drift curves, obtained with the NLDA, corresponded to stages of the system where the strength has radically dropped from its maximum, as shown with the pushover analyses. Furthermore, there was no basis to assume that a larger PGA of the input motion would imply larger demands be imposed upon the structure, unless the same input motion was used. As the same ground motion will almost assuredly never repeat itself in reality, finding a PGA associated to a collapse limit state for a given system does not appear to be a reliable parameter to establish that a given system behaves better than another (Quintana Gallo et al., 2017).

Using numerical simulations, Zhou et al. (2012, 2014b) studied the dynamic response of a timber and reinforced masonry core hybrid structure subject to earthquake ground motions. The description of this system and its preliminary experimental evaluation was presented in Zhou et al. (2014a). The numerical investigation was carried out using a 6-storey prototype structure whose plan layout is shown in Figure 9(a). The structure, assumed to be in Vancouver, Canada, was designed following the National Building Code of Canada. The building was modelled in two dimensions only to study its behaviour in the E-W direction. The model was constructed in the finite element computer program ABAQUS (2010) using macro-elements. The hysteresis of the timber and masonry walls, and the connections, were modelled with the rule proposed by Xu and Dolan (2009), which is a modified version of the Bouc-Wen rule (Wen, 1976). They were calibrated with experimental data (Zhou et al., 2012; Shedid et al., 2008). All the members of the same material were grouped and represented by super elements with equivalent properties, as shown in Figure 9(b).

For the NLDA, a set of 10 earthquake records were considered. These were obtained from the Pacific Earthquake Engineering Research Center database and considered events with a moment magnitude between 6.5 and 7.6 and stations located in soil type D. These input motions were selected such that the average acceleration response spectrum matched the design spectrum for Vancouver, Canada. The results of these analyses were evaluated in terms of the load shared by the RCM and timber in relation to those obtained for a timber-only benchmark structure. Three designs of the RCM wall were included in the investigated system, leading to three hybrid buildings: HB1, HB2 and HB3. The difference between these three buildings was that the RCM wall was designed to take 25%, 50% and 100% of the total base shear resisted by the complete hybrid system, respectively.

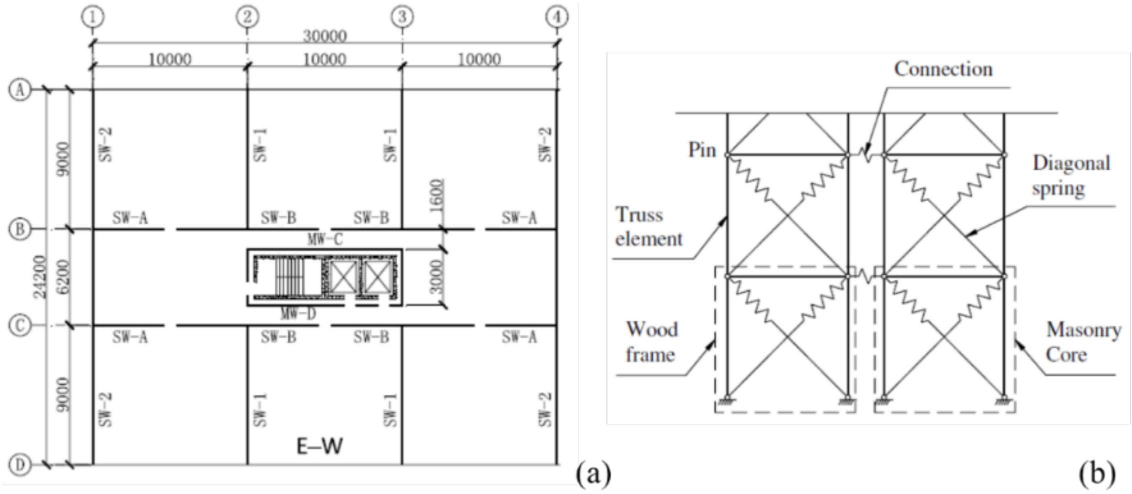


Figure 9. Numerical investigation on timber-RCM hybrid system: (a) prototype structure layout; (b) model description.

(Reproduced from Zhou et al., 2014b, with permission from ASCE.)

The results of all NLDA showed that, on average:

- the base shear obtained with the hybrid buildings HB1, HB2 and HB3 was approximately 120%, 140% and 180% of that obtained with timber-only model, respectively
- the base shear taken by the RCM wall with HB1, HB2 and HB3 was approximately 30%, 70% and 150% of that resisted by the timber walls, respectively
- the base shear resisted by the timber walls of buildings HB1, HB2 and HB3 was approximately 90%, 80% and 70% of that resisted by the walls of the timber-only benchmark, respectively.

The results also showed that the inclusion of the RCM wall significantly reduced the inter-storey drifts obtained with the timber-only structure and that, when the strength of the RCM increased, the inter-storey drift of the masonry structure decreased, as expected.

The larger shear force attracted by the hybrid buildings and the reduced inter-storey drifts obtained with the stronger RCM walls can be attributed to the increase in the stiffness of the system, as this is directly proportional to the strength (Priestley, 1993a). The larger shear attracted by the RCM walls within the hybrid buildings when compared to the timber counterparts was also attributable to the relatively larger stiffness and strength of the RCM wall in the former case. Finally, when comparing the value resisted by the timber walls of the hybrid buildings with those of the benchmark building, it was concluded that the inclusion of the RCM wall reduced the shear demands on the timber members. The authors acknowledged that this decrease was an average and that there were few cases when the shear demand in the timber walls of the benchmark was smaller than in the hybrid structure. As the RCM wall was stiffer than the timber members, it attracted more shear load, and the RCM wall increased the shear demand in the structure as it increased its stiffness. Therefore, a smaller demand in the timber members of the hybrid structure over the benchmark counterpart was not guaranteed.

The numerical investigation carried out by Zhou et al. (2014b) provided a preliminary understanding of the dynamic response of multi-storey buildings using the hybrid system proposed by the authors, whose main benefit was a reduction in the magnitude of the displacements. However, to more profoundly understand the effectiveness of this system in the dynamic range, it would be appropriate to conduct shake table experiments, where the damage of the structural elements can be directly assessed.

Fairhurst et al. (2014) presented numerical analyses of a hybrid system, initially proposed by Green and Karsh (2012), named finding the forest through the trees (FFTT). The FFTT system used CLT panels and glulam and steel prismatic members. CLT walls were used to control the lateral displacements and to resist most of the seismic shear demand. CLT was also used as diaphragms to constrain the vertical elements, providing structural integrity. Glulam was used in columns to primarily resist gravity loads.[5] Steel was used for the beams to resist gravity loads and to dissipate seismic energy via inelastic deformations. The latter implies a weak beam-strong column capacity design of the frames. In the concept proposal of this system (Green & Karsh, 2012), four options were presented (Figure 10), each of them suitable up to a certain number of storeys.

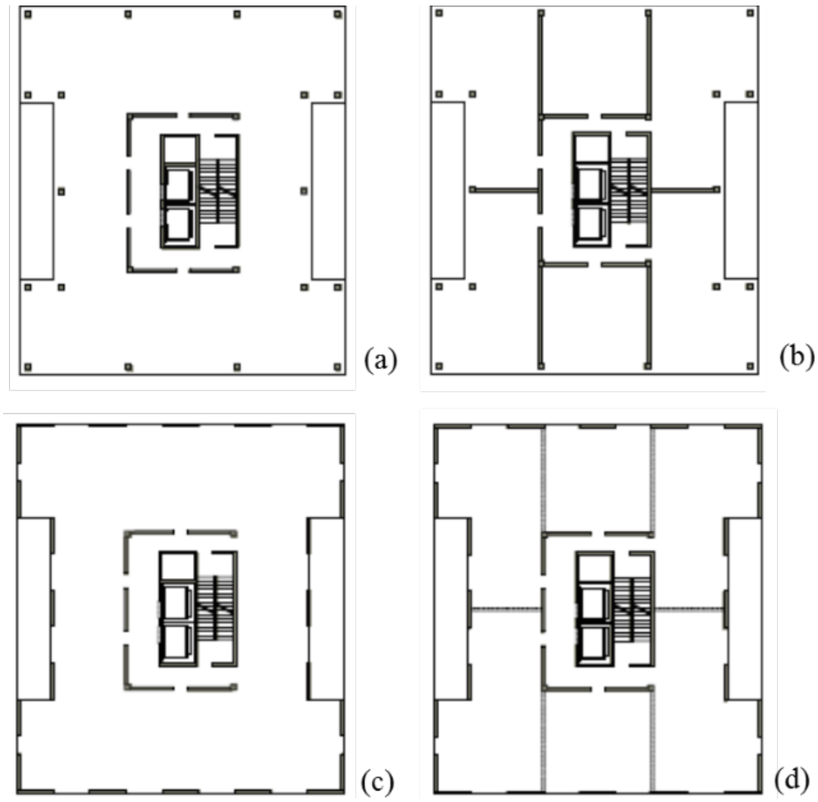


Figure 10. FFTT system: (a) option 1 – core walls and perimeter frames (up to 12 storeys); (b) option 2 – core and interior walls and perimeter frames (up to 20 storeys); (c) option 3 – core and exterior walls (up to 20 storeys); (d) core, interior and exterior walls (up to 30 storeys).

(Reproduced from Green & Karsh, 2012, under Creative Commons CC licence – Attribution Non-Commercial Share Alike.)

Fairhurst at al. (2014) modelled the four FFTT prototype buildings in SAP2000 (Computers and Structures Inc., 2004) and OpenSees (McKenna et al., 2000),

[5] It is important to note that there are no purely gravity-resisting elements in a building constructed in seismically active regions, as the elements will inevitably deform due to seismic actions when the structure moves.

including models for the hysteresis behaviour of CLT, glulam and steel elements and the connections. They conducted over 800 NLDA on 39 models using bidirectional ground motion excitation. The input included 10 ground motions (with two components each) recorded during different seismic events. The ground motions were used twice such that both components were imposed in each principal direction of the building plan, selecting the most demanding response for comparison.

The results of each NLDA were evaluated in terms of maximum inter-storey drift ratios (*dr*). For each building option and number of storeys, the mean value of the *dr* for each floor and set of 10 motions (using the most demanding orientation) was calculated. The results showed that, in all cases, *dr* remained below 1.0%. A limit of 2.5% was proposed as an acceptable maximum, based on the work done by others (Pei et al., 2013). However, this limit was not linked to any strain in the structural members, which would be a more reliable way of checking the maximum deflection that a structure can undertake prior to collapse. The authors also included a stricter limit of *dr* = 1.1%, based on the work done by Buchanan and Fairweather (1993). The roof drift ratio (RDR) was also evaluated, showing that this value was less than 1% in all cases.

The results also showed that the inelastic rotation at the ends of the steel beams were smaller than 50% of the ultimate for the life safety/collapse prevention limit state established by ASCE/SEI 41-06 *Seismic rehabilitation of existing buildings*. The base shear in all the models fell somewhere between one-third and one-half of the base shear obtained with the elastic spectrum required by the National Building Code of Canada.

The FFTT system provides a viable alternative for constructing timber-based hybrid systems, with the CLT walls providing stiffness and the steel beams a source of seismic energy dissipation. However, there are several points that seem to require further investigation and clarification. The results of the analyses were average values only, and there should be some cases where the predicted values were larger than the one presented. Hence, the scatter in the results should also be discussed when addressing the problem from a statistical perspective. In terms of the mechanics of the system, it is questionable that the steel beams could experience plastic hinging as the CLT diaphragm will prevent them from deforming as if they were detached. Therefore, one could expect that there would be much more energy dissipated in the connections of the CLT panels than anticipated, which can be detrimental for the structural integrity of a timber system in general. Hence, more experimental research to address these issues is needed.

Gilbert et al. (2015) proposed the use of heavy timber frames (glulam) with dissipating steel braces (BRBF) and a reduced-section beam steel connection as energy dissipating elements (DMRF).

The main purpose of the new system was to avoid damage occurring in the connections between timber members, as they are prone to bearing failure, excessive fastener yielding and/or splitting perpendicular to the grain. To study the advantage of hybrid over traditional steel frames, the authors conducted non-linear dynamic analyses on 6-storey prototype buildings, using planar (2D) models implemented in the computer program OpenSees (McKenna et al., 2000). A suite of 44 scaled earthquake ground motions was used as input for the NLDA. These were based on the far-field records suggested by FEMA P695 (Applied Technology Council, 2009).

The results of the NLDA showed that, for all the records, all the building models with the different systems did not reach nominal collapse,[6] except for the case of one ground motion, when all the models reached that stage. Consequently, this input was removed from the suite of records to continue the investigation. The results also showed that, on average, the performance of the hybrid and steel-only buildings were similar in terms of maximum inter-storey drift ratios and floor accelerations. The mean of the maximum inter-storey drifts of all analyses for each structure remained below 2.5%, the maximum allowed by the National Building Code of Canada.

The structures with BRBF, steel-only and hybrid, presented similar responses, with a maximum mean inter-storey drift ratio (*dr*) of 1.1% and 1.2%, respectively. The models with moment-resisting frames only, steel-only and hybrid, also presented similar global responses with maximum mean *dr* of 2.5% and 2.4%, respectively. The maximum average residual drifts of both braced structures were smaller than the corresponding drift of the unbraced counterparts. The mean floor acceleration in all storeys and models remained below 0.5 g, and Gilbert et al. (2015) concluded that the research they conducted showed that the implementation of ductile steel systems within timber structures could overcome the disadvantages of timber-only seismic-resistant structures. They also concluded that designing these hybrid structures with larger reduction factors of the acceleration spectrum, *R*, resulted in similar seismic performance compared to their steel-only counterparts.

The work presented by Gilbert et al. (2015) is relevant as it addresses the problem of energy dissipation sources in timber structures. They propose, for the first time, the use of buckling restrained braces within these structures as well as connections with steel fuses at the end of the beams. However, the work relied on numerical analyses only. As such, it provided a general idea of the improvement that these techniques may represent for timber buildings but one that needs to be tested against experimental evidence. It is important to note that some of the authors did present a paper with experimental research on the topic, but they used steel braces with friction dissipaters instead of BRBs (Gilbert and Erochko, 2016), as previously discussed in this report. As such, the experimental testing of BRBs within massive timber construction represents a gap in the literature that deserves attention.

Goertz et al. (2016) proposed a hybrid system that included CLT panels and ductile steel plate connections, as shown in Figure 11. The system also considered the use of CLT diaphragms and steel frames. The aim of the proposed hybrid system was to use the large stiffness of the CLT panel to resist the seismic actions and the steel to provide a stable and reliable source of energy dissipation. The steel plates were designed to run along the entire height of the wall, connected to the CLT panels via screws (see Figure 11(b)). The steel plates were spliced at the floor level, as shown in Figure 11(d), and they were detached from the floor diaphragm.

Goertz et al. (2016) evaluated the system using numerical analyses of a prototype structure (see Figure 11(a)) implemented in SAP2000 (Computers and Structures Inc., 2004). The CLT panels were modelled as orthotropic shell elements with equivalent stiffness calculated as recommended in the FPInnovations CLT handbook (Gagnon & Pirvu, 2011), and the steel plates were modelled with frame elements. The connections were modelled with linear-elastic (hold-downs and brackets) and non-linear (t-stub, see Figure 11(d)) spring elements. The hysteretic behaviour of the t-stub connection was modelled with a pivot hysteresis rule (Dowell et al., 1998). The model was

[6] The authors do not mention the criteria for the determination of the collapse of the model.

subjected to bidirectional earthquake ground motion demands. These input motions were selected using the conditional mean spectrum method (Baker, 2011), adapted as per Atkinson et al. (2011), resulting in a set of 10 records (with two components each).

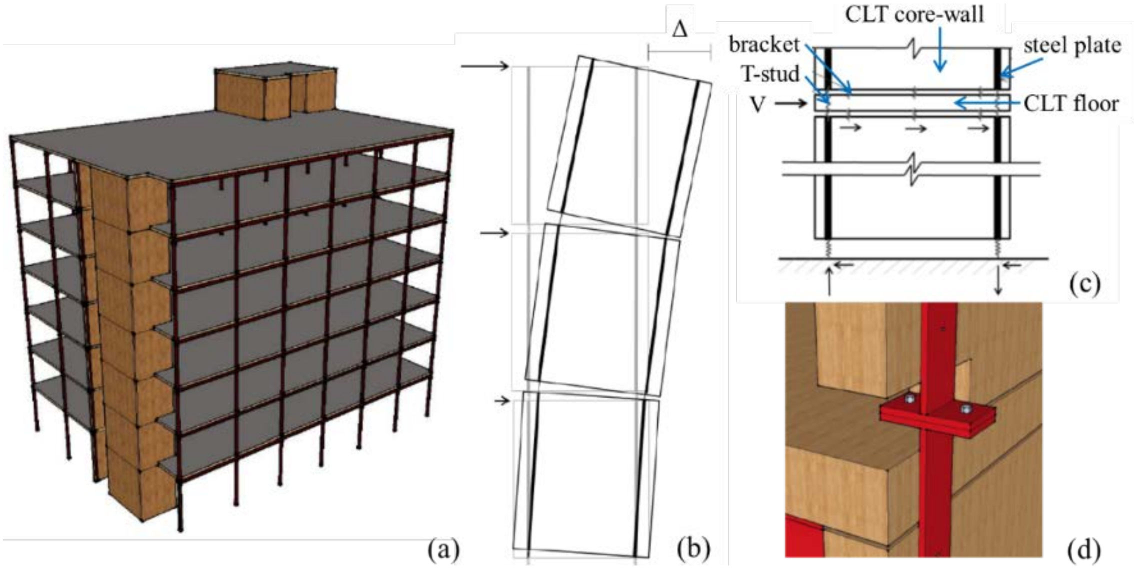


Figure 11. CLT panels with continuous steel plate connections: (a) hybrid building vision; (b) CLT hybrid wall concept; (c) detail of connection between wall and diaphragm; (d) detail of steel splicing.

(Reproduced from Goertz et al., 2016, with permission from the authors.)

The results of these analyses showed that the roof drifts did not exceed 1.5% of the height in both directions in all cases. The inter-storey drift ratios did not exceed 1% in all storeys and all records, except for two cases, where the *dr* was close to 2% in some storeys. Hence, the authors noted that average values of the *dr* obtained with all the records would not provide a realistic hazard level for the whole set as these two were significantly more damaging (demanding) than the rest (Goertz et al., 2016).

The hybrid wall system proposed by Goertz et al. (2016) represented a viable alternative for providing ductility to CLT panels as it provided a new source of energy dissipation. However, more focus should be placed on the robustness of this system. If the vertical steel plates or the t-stub connections between them experience excessive damage or break, they could potentially compromise the integrity of the entire wall and building. In addition, the system needs experimental investigations to produce empirical evidence related to its behaviour, bearing in mind that, even then, there is no definite validation of a system or theory in empirical science per se (Popper, 1963).

Marin and He (2016) carried out numerical simulations of a prototype hybrid structure with steel frames, timber shear walls and timber diaphragms. They used two configurations for the timber walls, one placed in the perimeter and the other placed in internal bays, as shown in

(a) and (b), respectively. The predicted response of the hybrid structures was compared with that of a steel frames-only building and timber diaphragms. This structure had steel frames in all the bays of the structural layout shown in

.

The three prototype buildings were modelled in the computer program ABAQUS (2010). The steel members were modelled with frame elements and the timber walls with equivalent diagonal springs. A set of nine input motions was used in each direction. These inputs were based on the NS and EW components of the ground motion recorded at El Centro station during the Imperial Valley earthquake (California 1940) and one accelerogram representative of a ground motion associated to the Chinese city of Shanghai.[7] These motions were scaled in amplitude to reach 0.07 g, 0.20 g, and 0.40 g, as required by the Chinese Code of Buildings (Marin & He, 2016).

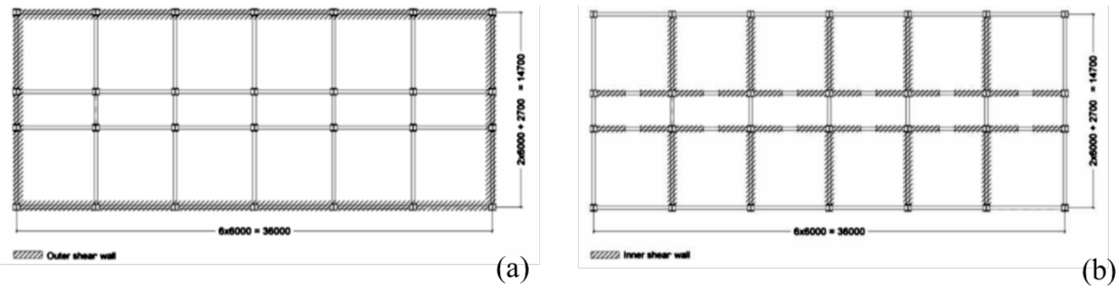


Figure 12. Steel frames and timber shear walls: (a) external walls; (b) internal walls.

(Reproduced from Marin & He, 2016, with permission from the authors.)

The results of the NLDA showed that:

- the inclusion of the LTF walls increased the floor accelerations at different storeys, with the largest values being predicted with the hybrid structure with internal walls
- the storey displacements and inter-storey drift ratios decreased with the inclusion of the LTF walls, particularly when the LTF walls were placed in the interior
- the base shear increased with both hybrid structures when compared to the bare steel frame.

3.3 Analyses at a component level

Dickof et al. (2014) proposed a hybrid system consisting of steel moment-resisting frames infilled with CLT panels, connected via steel brackets nailed to the CLT and bolted to the steel, as shown in Figure 13.

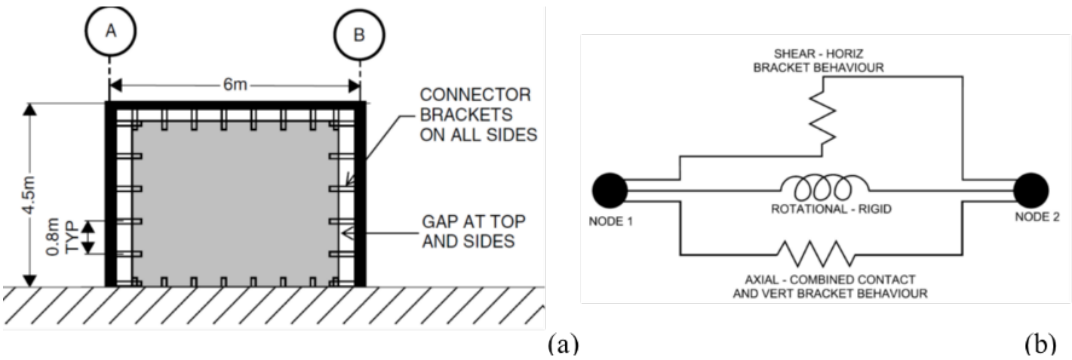


Figure 13. Steel frame infilled with CLT panel: (a) single-storey, single-span hybrid system; (b) numerical model of the connector brackets.

(Reproduced from Dickof et al., 2014, with permission from the authors.)

[7] The authors do not provide background for such affirmation.

Using a numerical model implemented in OpenSees (McKenna et al., 2000) and monotonic pushover analyses, the authors conducted a parametric study of a single-storey, one-bay hybrid structure to evaluate the effect of the panel thickness, crushing strength and confinement gap on the system response. Following the evaluation of the single hybrid wall panel, the authors modelled a three-bay structure and 3, 6 and 9-storey structures with different panel configurations (one, two or three bays infilled) and performed monotonic pushover analyses.

The inelastic properties of the steel beams/columns and the brackets were modelled with the rule proposed by Lignos and Krawinkler (2011) and the OpenSees Pinching4 rule (McKenna et al., 2000), respectively. The CLT panels were modelled with linear elastic shell elements.

The parametric study of the single-bay structure showed that an increase in the ultimate strength of the CLT and the thickness of the panel increase the strength of the system, particularly after the yielding point. An increasing size of the gap, on the other hand, was found to have a negative influence on the strength of the system, achieving the best behaviour with only 3 mm, as can be observed in one of the figures presented by Dickof et al. (2014). Nevertheless, the authors proposed that increasing the size of the gap increased the ductility of the panel.

The study on the multi-storey structures revealed that:

- increasing the number of infilled panels increased the strength and stiffness of the system, with a minimal decrease in the deformation capability
- the ductility of the steel frame had little influence on the ductility of the system
- a characteristic ductility factor for the system was 2.5
- the over-strength factor of the system can be taken as 1.25.

Although the authors present a novel hybrid structure, there are limitations that should be recognised. Firstly, gravity loads were not considered in the investigation. If they were included, they may affect the behaviour of the connection brackets and the size of the steel elements, for example, as the CLT panel would not carry most of that load. There is a need for considering the cyclic behaviour of the system, as it was deemed to withstand seismic demands. More importantly, the use of pushover analyses can be quite limited. As discussed by Krawinkler and Seneviratna (1998), a carefully performed pushover can provide insight into aspects that control the behaviour of a structure vibrating in the fundamental mode. However, the deformation estimates obtained with this method can be very inaccurate for structures in which the higher modes have a strong influence on the dynamic response (Quintana Gallo, 2008). More critically, there is a concern that a pushover analysis may only detect the first mechanism that forms when a structure deforms in the fundamental mode of vibration but may not expose other weaknesses that can develop after the dynamic characteristics of the system change while being affected by the seismic input (Krawinkler & Seneviratna, 1998). Finally, the experimental testing of the system is necessary for contrasting the numerical conjectures in any case.

Li et al. (2014) performed numerical studies on the hybrid system proposed by He and Li (2012) and He et al. (2014). They developed a numerical model of the hybrid timber-steel walls and implemented it in the computer program ABAQUS (2010). This model was based on one proposed by Gu and Lam (2004) for nail connections and wood shear walls. The model was calibrated with experimental results (He & Li, 2012; He et al., 2014) and used to conduct parametric studies.

It was found that the predictions of the numerical model agreed reasonably well with the test results for single and double-sheathed specimens (see He & Li, 2012; He et al., 2014). Nevertheless, the model did not predict the strength degradation that occurred during the last experimental cycles due to the fracture of the weld in the beam-column joint connection not included in the model. The lateral load shared by the steel frame and the LTF infill was also numerically evaluated. The results of the simulations showed that the model captured the experimental results in these terms reasonably well.

The results of the parametric analyses showed that:

- the initial lateral stiffness of the hybrid shear wall increased when the double-sheathed LTF walls were used
- the instantaneous lateral stiffness of the system with single and double-sheathed infills converged to similar values at increased values, due to the degradation in the stiffness and strength
- at small drift ratios (less than 0.5%), the load was primarily carried by the LTF panel.

The model developed by Li et al. (2014) provided a valuable contribution for the numerical representation of the hybrid system under investigation as it provided good prediction capability. Nevertheless, it would be interesting to know how the model was calibrated and if there was any tuning of the parameters of the model after contrasting the prediction against the experimental results. As noted previously, the prediction using these types of macro models can sometimes be very accurate and not so much in others, requiring feedback from experiments to acceptably capture their results (Quintana Gallo, 2014). As such, care should be placed in the selection of the parameters governing the model.

Li et al. (2015) used the model described above (Li et al., 2014) to perform additional parametric studies and non-linear dynamic analyses to further evaluate the hybrid system proposed by He and Li (2012) and He et al. (2014). Through the parametric investigation they found that:

- the effectiveness of the hybrid system was directly proportional to the infill-to-frame stiffness ratio (R_{if})
- the effectiveness of the infill wall decreased when the spacing of the bolts in the wood-steel connection was increased, as it reduced the capability of transferring the shear loads between both parts of the system
- the use of stiffer connections between steel beams and columns resulted in larger initial stiffness and ultimate load capacity of the system
- the stiffness and the strength of the system decreased with an increase in the height of the wall
- the infill carried more load and dissipated more energy with greater wall lengths.

In the non-linear dynamic analyses (NLDA), the system was modelled with and without infill panels, representing a hybrid wall and a bare steel frame, respectively. Three earthquake ground motions were used as input, each of them scaled to have acceleration amplitudes (PGA) associated to three return periods (probabilities of exceedance). Per Li et al. (2015), the results showed that the timber infill carried the largest part of the load during minor earthquakes and dissipated an important part of the energy during major earthquakes. Perhaps more importantly, it was found that the inclusion of the infill panel reduced the maximum displacement of the system in all cases. It was concluded that the system members showed a complementary hybrid effect and provided robustness to the hybrid elements.

The additional parametric analyses presented by the authors provided valuable findings and were further supported by the non-linear analyses. However, the conclusion of the increasing robustness was not supported by the findings, revealing the need for shake table experiments to address the issue.

Hashemi et al. (2016) proposed a steel-CLT hybrid wall system with friction connections and horizontal post-tensioning (see Figure 14). The system was aimed at providing stiffness, ductility, energy dissipation and self-centring capacity to a standard timber system. It was similar to the heavy timber walls with hybrid connections previously presented by Buchanan et al. (2008), which in turn was based on the PRESSS technology proposed for precast RC members by Priestley (1993b).

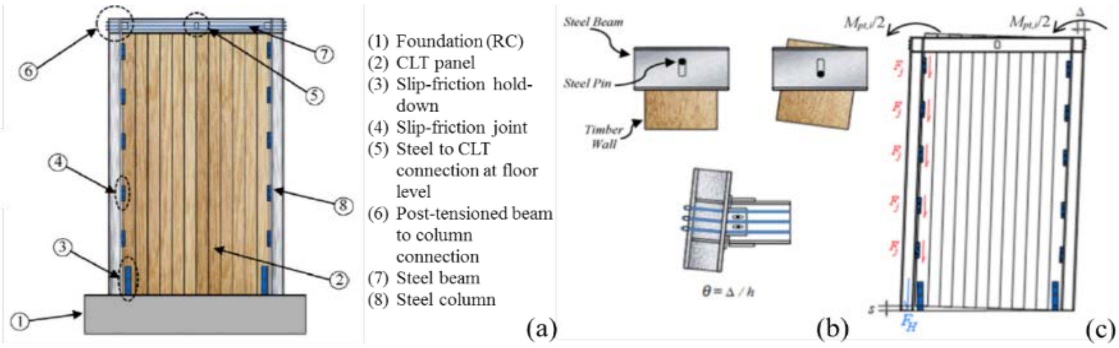


Figure 14. CLT and steel rocking wall: (a) components; (b) connection details; (c) wall lateral mechanism.

(Reprinted from Hashemi et al., 2016, copyright (2016), with permission from Elsevier.)

The hybrid wall included:

- a CLT infill panel
- steel columns and beam around the perimeter of the CLT panel
- slip-friction connections between the CLT panel and the steel frame
- slip-friction hold-downs between the CLT panel and the foundation, normally made of RC
- a special beam to CLT connection (see Figure 14(b))
- horizontal post-tensioning of the steel beam-column connection.

The special connection at the centre of the beam span was designed to accommodate displacement incompatibilities between the wall and the diaphragm, as it allowed for vertical relative movement but imposed displacement compatibility in the horizontal direction.

To investigate the efficiency of the new system, Hashemi et al. (2016) simulated the cyclic displacement-controlled hysteretic behaviour and the dynamic response against earthquake ground motions of a prototype hybrid wall using a numerical model implemented in the computer program SAP2000 (Computers and Structures Inc., 2004). The CLT panel was modelled with non-linear layered shell elements and the slip friction devices and post-tensioning with multi-linear plastic links.

The model of the hybrid wall was firstly subjected to quasi-static cyclic lateral loading at increasing amplitudes, with a maximum displacement drift of 3.75%. The resulting load-displacement loops resembled the shape of a flag, typical of self-centring systems such as in the PRESSS technology (Priestley & Tao, 1993; Priestley et al., 1999).

For the NLDA, the model was modified by replicating the hybrid wall in the orthogonal direction. A set of five ground motions, modified to match two given design spectra (R = 1.0 and R = 1.8, respectively, as per NZS 1170.5:2004 *Structural design actions – Part 5: Earthquake actions – New Zealand*, were used as input in both principal horizontal directions at the same time.[8] A viscous damping of 2% of the critical value was assigned to all the modes of vibration, following the approach proposed by Wilson and Penzien (1972) and recommended by Carr (2016a).

The results showed that:

- in all cases, the maximum roof drift ratio (RDR) was below the design value of 3.75%
- in all cases, the residual RDR was close to zero
- the maximum floor accelerations at the roof level were less than 1 g, with an overall maximum of 0.91 g, which is much lower than the experimental value recorded during the shake table tests performed by Ceccotti et al. (2013)
- the maximum simulated base shear force was, in all cases, smaller than the design value for ultimate limit state (ULS), but it exceeded that threshold in all cases, with the absolute maximum being 1.8 times the design value recommended by others (Kelly, 2009).

3.4 Summary

Most of the investigations presented above have proved the advantages of different timber-based hybrid systems. A summary is presented in Table 2.

In general, the investigations were carried out at a system level, i.e. using a model of a whole building that includes members made of timber and other materials. A few were conducted at a component level, mostly to compare the numerical predictions with the results of experiments.

All the analyses included elements with non-linear properties. All the inelastic properties of the structural members were modelled with line elements, except for Hashemi et al. (2016), who used fibre elements. The majority of the research included dynamic analyses, whereas only a few included pushover studies.

Despite the undeniable importance of numerical investigations in seismic engineering, it must be emphasised that these are meant to provide preliminary information about the performance of structures subjected to seismic actions and not definitive answers. As mentioned during the body of this section, the models implemented cannot be validated, as many researchers believe, because that is philosophically not possible in empirical science. Furthermore, the results of these investigations might significantly differ from what would occur in reality as has been found previously when predicting the dynamic response of experimental models under simplified and controlled laboratory conditions (Quintana Gallo, 2014). Nevertheless, these results can be useful to estimate relative advantages and disadvantages of one system over another. In this

[8] Note that the same recorded component to the motion was used in both directions and not the corresponding components in each direction, as is typically done.

case, this is applicable to comparisons between a structure designed using a timber-only or a hybrid solution.

Table 2. Summary of numerical investigations.

Authors	Timber	Other system	Modelling approach	Analysis type	Model level
Asiz et al., 2011	LTF walls	Steel frame	2D-macro	Pushover, NLDA	System
Zhou et al., 2014b	LTF walls	RC masonry	2D-macro	NLDA	System
Fairhurst et al., 2014; Green & Karsh, 2012	CLT panels; glulam columns	Steel beams	2D-macro	NLDA	System
Dickof et al., 2014	CLT panels	Steel frames	2D-macro	Pushover	Component
Li et al., 2014	LTF walls	Steel frames	2D-macro	Cyclic pushover	Component
Li et al., 2015	LTF walls	Steel frames	2D-macro	NLDA	Component
Gilbert et al., 2015	Glulam frames	Steel buckling restrained braces	2D-macro	NLDA	System
Gilbert et al., 2015	Glulam frames	Reduced-section steel beam	2D-macro	NLDA	System
Goertz et al., 2016	CLT panels	Steel plate connections	3D-macro	NLDA	System
Marin & He, 2016	LTF walls	Steel frames	2D-macro	NLDA	System
Hashemi et al., 2016	CLT panels	Steel frames; steel dissipaters	2D-fibre	Cyclic pushover; NLDA	Component

4. Connections

4.1 Introduction

The adequate design of the connections between structural elements in modular construction is a critical aspect of seismic engineering. During the 1994 Northridge earthquake in California, for example, several buildings in the Los Angeles area experienced severe damage in the beam-column joint connections of moment-resisting frames even under rather limited inelastic demands (Krawinkler, 1996; Hamburger et al., 2009). Consequently, the American provisions for the seismic design of steel connections adopted new safety factors and limit states to be checked for their design to ensure a larger relative strength of the connections in comparison to the rest of the structural members (i.e. ANSI/AISC 341-10 and ANSI/AISC 358-16 *Prequalified connections for special and intermediate steel moment frames for seismic applications*).

These connecting elements also play a fundamental role for the seismic behaviour of timber and timber-based structures. However, in current practice, the connections of timber structures are expected to be damaged as they constitute the primary source of energy dissipation. This is extremely relevant, as it is foreseen that this is not an optimal solution because damage of the connections can jeopardise the integrity of any structure, including timber and timber-based hybrid buildings.

This section presents a concise summary of relevant publications on connection and connection systems for timber-based hybrid buildings, as this topic merits a comprehensive literature review for itself.

4.2 Examples of connections in hybrid construction

Bainbridge and Mettem (1998) presented a review of moment-resisting connections for heavy timber frame structures. They compared traditional exposed and concealed connections, arguing that the use of exposed steel plates with nails and rods can result in unsightly, bulky and inefficient connections for large structures, which detracts from the aesthetic appeal of timber as a material. Concealed steel plate connections on the other hand, offer advantages such as superior aesthetics and resistance against fire and environmental degradation while also providing a wider scope for prefabrication and the opportunity for improved quality control.

Bainbridge and Mettem (1998) divided traditional connections for heavy timber into the categories of:

- mechanical connections – dowels, staples, bolts, nails, screws, split rings, shear plates, nail plates and so on
- adhesive connections – high-strength polymeric resins often used with bonded-in bars or plates
- carpentry connections – traditional mortise and tenon joints, dovetail joints and so on.

The authors noted that carpentry connections were out of the scope of their review, as they generally lacked the necessary strength and moment-carrying capacities.

Concealed connections, in turn, can be divided into the categories of:

- concealed bonded-in rods

- concealed bonded-in plates
- adhesive bonded surface contact joints
- timber connectors within lapped dowel-type joints
- dowel-type joints.

Figure 15 shows examples of these concealed connections.

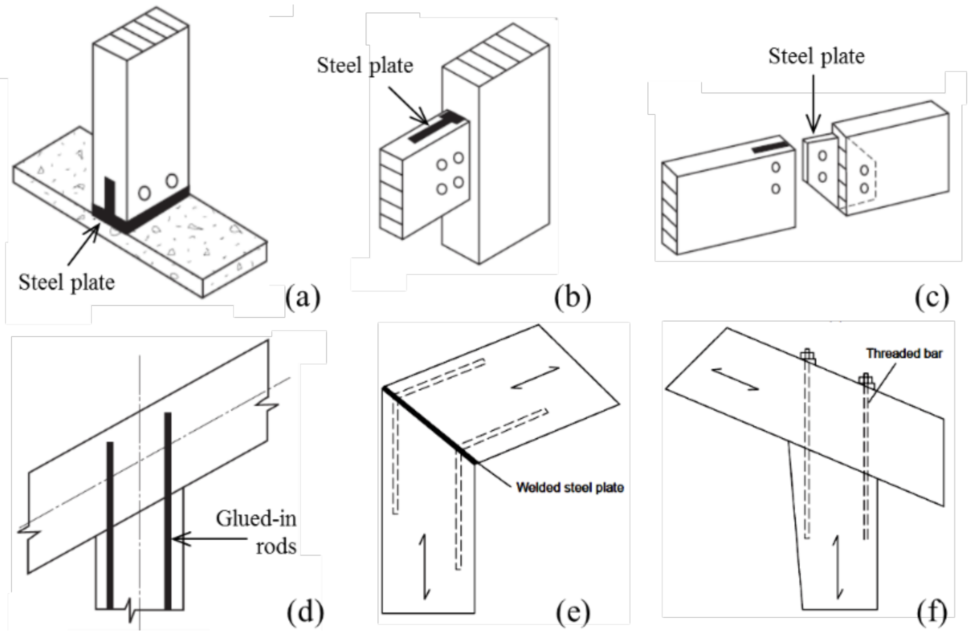


Figure 15. Concealed traditional timber connections: (a) concrete footing-to-column mechanical; (b) mechanical beam-to-column; (c) mechanical beam-to-beam; (d) column-to-beam glued in rods; (e) beam-to-column glued-in and welded; (f) column-to-beam mechanical with threaded bars.

(Republished with permission from the Institution of Civil Engineers from Bainbridge & Mettem, 1998; permission conveyed through Copyright Clearance Center, Inc.)

Gattesco and Toffolo (2004) tested traditional mechanical connections with concealed steel plates for glulam timber elements, loaded in the direction parallel and perpendicular to the grain. The specimens tested by the authors are presented in Figure 16.

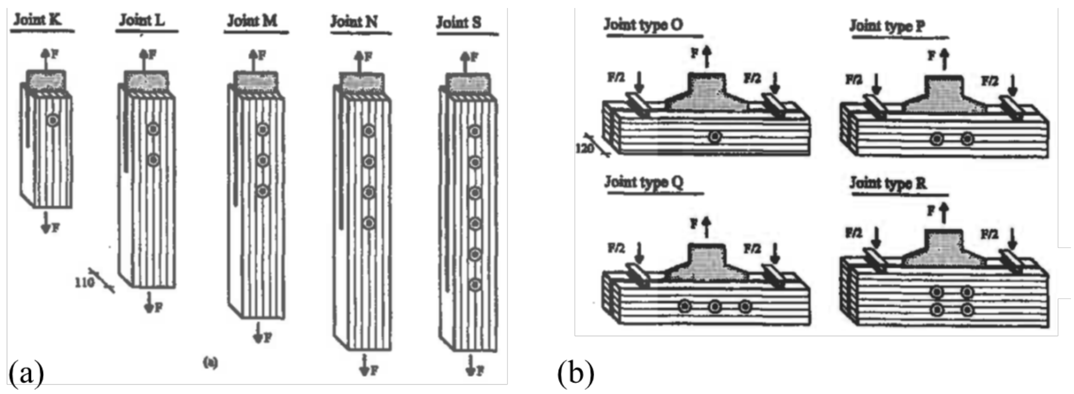


Figure 16. Glulam-to-steel bolted connections: (a) specimens for parallel-to-grain testing; (b) specimens for perpendicular-to-grain testing.

(Reprinted with permission from Springer International Publishing AG, Springer Nature, from Gattesco & Toffolo, 2004.)

The glulam specimens were made from 33 mm thick timber laminae glued with resorcinol adhesive. The connections consisted of concealed steel plates anchored to the glulam using one to six bolts and one to four bolts for specimens loaded parallel (S1) and perpendicular to the grain direction (S2), respectively.

The test procedure included a monotonic displacement-controlled quasi-static loading regime only. The load-slip results of specimens S1 showed limited ductility for specimens with more than one anchor, sometimes failing before the artificial yielding point was reached. The load-slip behaviour of specimens S2 were also characterised by limited (if any) ductility. However, better results were achieved with one fastener and with configuration R (see Figure 16(b)).

Inoue et al. (2004) presented a study on the seismic performance of connections between timber and RC elements as part of a Japanese project on hybrid structures that started in 1999 (Sakamoto et al., 2004). They carried out an experimental investigation using Sugi (a Japanese timber species) glulam timber columns connected via glued-in lag screws, metal connectors and deformed steel bars to an RC foundation beam. A different number of connectors was used for each type (two, four and eight), resulting in a total of nine specimens. These specimens were subjected to a quasi-static cyclic lateral loading protocol applied at the top of the column, while the concrete footing was fixed to the strong floor.

The results of the specimen connected with steel deformed bars showed that, when two and four were used, the bars were pulled out of the timber, whereas when eight of these bars were used, prying failure out of the concrete was observed (see Eligehausen et al., 2006). The results of the tests of the specimens with metal connectors showed that, when two, four and eight connectors were used, adhesive failure, adhesive and concrete failure and concrete prying failure modes occurred, respectively. Finally, during all the experiments of the specimens with lag screws, pull-out failure in the timber was observed. Given the non-ductile failure modes achieved in all the experiments (i.e. bar pull-out from the timber and concrete prying out), these connections were all considered to be poorly performing.

Asiz and Smith (2011) investigated connections for hybrid structures involving CLT diaphragms and steel frames and joists using simple fastening techniques such as screws (see Figure 17(a)). They conducted experiments with the set-up shown in Figure 17(a) and (b), following the test methods described in ASTM D5652-95(2007) *Standard test methods for bolted connections in wood and wood-base products*, which involves a monotonic quasi-static loading protocol.

The CLT panel was placed in two positions such that the major axis direction, defined as that with the greater number of layers parallel to it, was parallel or perpendicular to the direction of loading, as shown in Figure 17(b) and (c), respectively. The CLT panel was five-layered and 150 mm thick (in total). The steel member was a W8x24, as defined by the American Institute for Steel Construction design manual (AISC, 2006), representing a floor joist. These members were joined with four screws placed in the configuration as shown in Figure 17, made of three different types – a long SFS-screw (S1), a short SFS-screw (S2) and a lag screw (S3).

The steel flanges were predrilled to replicate typical construction practices, whilst the CLT was directly perforated with the screws. Washers were used with the steel to avoid pulling through the hole. Each test was replicated three times to understand the scatter involved in the results.

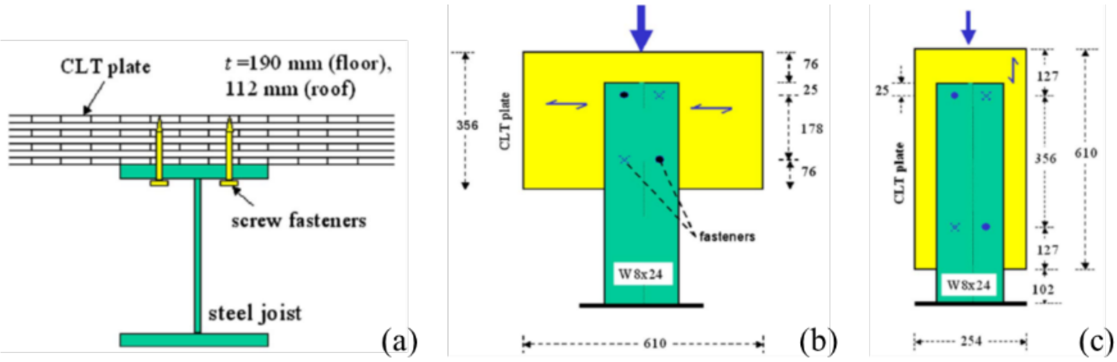


Figure 17. CLT-to-steel connection: (a) CLT diaphragm over steel beam connected via screws; (b) test set-up parallel to the major axis of CLT plate; (c) test set-up perpendicular to the major axis of CLT plate.

(Reproduced from Asiz & Smith, 2011, with permission from the Canadian Society for Civil Engineering.)

The results showed that, regardless of the loading direction and the type of screw, a ductile failure mode characterised by large post-elastic bending of the fasteners and some crushing of the CLT was obtained in all the tests. However, the connection was stronger and less stiff when the CLT was loaded perpendicular to the major axis. The use of longer SFS screws (S1 instead of S2) led to a greater strength and stiffness of the connection. Additional tests with type S1 screws with half of the previous spacing were conducted, showing 10% and 40% reduction of the stiffness and strength of the connection, respectively. Asiz and Smith (2011) concluded that the use of these types of screws to connect CLT to steel members is highly feasible.

This work presented the viability of a simple connection for hybrid structures combining CLT panels with steel beams. The results were promising, but further investigation is needed to account for cyclic-controlled and dynamic (uncontrolled, i.e. shake table testing) behaviour.

Schneider et al. (2014) proposed a steel tube connector for timber-based hybrid structures consisting of hollow steel tubes inserted inside CLT panel members. The connection was aimed at fulfilling seismic performance requirements and avoiding previously identified undesired failure modes of other types of connectors. To achieve this, the authors mentioned the need for:

- large initial stiffness, strength and ductility
- ease of manufacturing, installing, inspecting and replacing
- fire safety
- zero to minimal destructive influence on the timber member.

The steel tube connection consisted of a steel tube, a threaded rod of 12.7 mm diameter, a coupler and two nuts (for the 12.7 mm rod). In this case, the tubes were 50.8 mm, 76.2 mm and 101.6 mm in diameter and 99 mm in length, which was the thickness of the CLT panel.

As shown in Figure 18(b), the tube had a 25.4 mm hole on one point of its surface and a coupler in the opposite face pointing towards the centre of the hole. The installation of the tube required pre-drilling of the CLT panel and the part of the steel member where it will be connected (see Figure 18(a)).

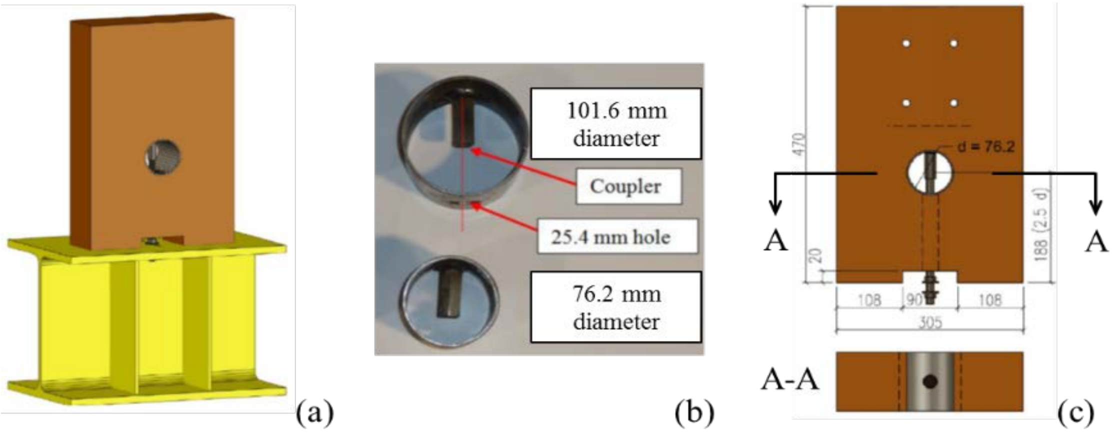


Figure 18. Steel-tube connection for hybrid structures: (a) test specimen; (b) steel-tube device; (c) detail of connector inside CLT.

(Reproduced from Schneider et al., 2014, with permission from the authors.)

Schneider et al. (2014) tested component samples like those presented in Figure 18(a) under monotonic and cyclic quasi-static loading protocols. The results of the monotonic tests showed that the connectors provided ductile behaviour and avoided any damage (cracking or crushing) to the CLT panel in all cases. Nevertheless, optimum behaviour was achieved with the 75 mm (3-inch) diameter tube, as it avoided strength degradation for a larger displacement range. The results of the cyclic tests, on the other hand, showed that, in all cases, the strength reached during the monotonic tests was not achieved, and degradation started at smaller displacements. The predominant failure mode observed was fracturing of the weld between the coupler and the tube after repeated yielding of the tube. However, the CLT panels were undamaged.

The research presented by Schneider et al. (2014) had the main advantage of protecting the CLT panel against damage. However, it is necessary to conduct experiments at a system level to better understand how connections perform within a structure. An obvious downside of the connector is that it cannot be aligned with the centre of a steel beam with an I-section (the most common cross-section in steel construction) due to the presence of the web. Hence, the CLT panel might induce torsional demands upon the steel beam in that case. Moreover, as discussed previously in this report, imposing the energy fuse in the connections may result in loss of structural integrity.

Hassanieh et al. (2016) investigated the short-term mechanical response of steel-to-timber connections. They studied the load-slip behaviour and failure modes of the connection between cross-banded LVL panels and steel beams using coach screws and bolts with and without adhesives via push-out tests. Steel nail plates were also introduced as a connection variable. An example of the test set-up and the concept for applicability are presented in Figure 19.

Hassanieh et al. (2016) distinguished three failure modes of the connections:

- Timber crushing with rigid body rotation of the screw (M1).
- Timber crushing with flexural hinging of the steel screw in one zone (M2).
- Timber crushing with flexural hinging of the steel crew in two zones (M3).

It was experimentally demonstrated that the predominant failure mode depended on the diameter and the embedded length of the screw inside the timber. For 8 mm and

12 mm fasteners, the dominant failure mode was M3 and M2 or M3, whereas for 16 mm and 20 mm screws, M2 was predominant.

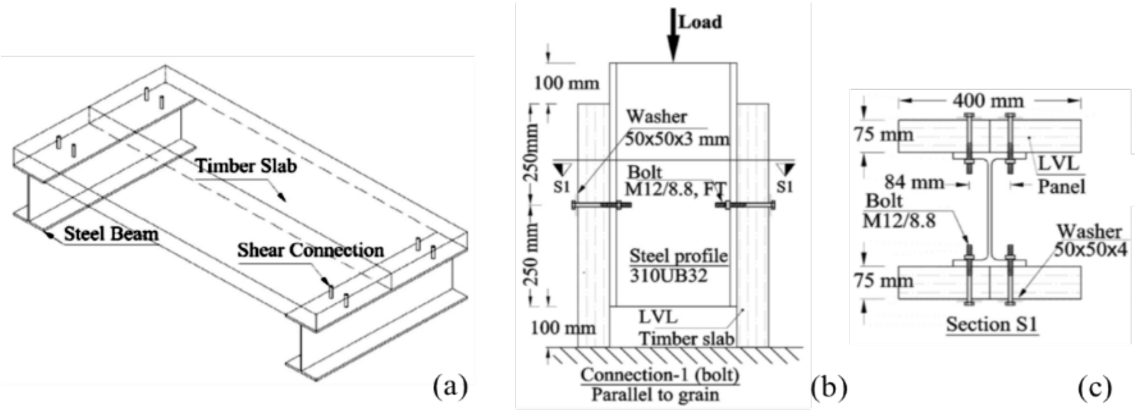


Figure 19. LVL-to-steel connections: (a) concept for applicability; (b) example of test set-up; (c) cross-section S1 as defined in (b).

(Reprinted from Hassanieh et al., 2016, copyright (2016), with permission from Elsevier.)

It was observed that the push-out behaviour of the joints with screw connectors was relatively ductile, whereas the behaviour of these joints with bolted connectors was rather brittle. However, when a steel nail plate was used in addition to the screws, the behaviour turned relatively brittle when loaded parallel and perpendicular to the grain directions. Testing of the specimens with glued screws also showed a brittle failure mode associated with fracture of the adhesive with no apparent damage to the LVL – named pull-through failure mode in anchorages for concrete (Eligehausen et al., 2006). Crushing of the LVL was limited to the vicinity of the screw or bolt when loaded parallel to the grain direction, whereas it propagated to a relatively large area when loaded perpendicular to the grain direction.

Hassanieh et al. (2016) did not provide conclusions on the experimental campaign, as they used the results to propose empirical equations to predict the load-slip behaviour of the connections. However, from their results, it was suggested that, if traditional connections were considered, large diameter screws without adhesives and nail plates would be preferred. This research was limited in that it did not address the cyclic load-slip behaviour of the connection in the quasi-static and dynamic ranges.

Xu et al. (2015) investigated the behaviour of dowelled steel-to-timber moment-resisting connections with two configurations. They used the results of these tests to evaluate the accuracy of a detailed three-dimensional finite element model with micro elements. Figure 20 shows the experimental set-up, the two dowel configurations considered for the LVL to steel plate joint (A and B) and the connection between the steel plate and beam. Four specimens were tested in total, two of each with configurations A and B, respectively. The testing protocol included quasi-static cyclic loading with two cycles only. The results of the experiments showed that the joints with configuration A presented a ductile failure mode, with plastic flexural deformations of the dowels and timber embedment. They also showed that the joints with configuration B were less ductile, as after significant bending deformations of the connectors, splitting of the timber member in the direction parallel to the grain was observed. The panel zone could undertake ductile rotations up to 5° and 3° with configurations A and B, respectively, about five and three times the yielding rotation

(1°). This indicated that the panel zone was strong enough to allow for inelasticity to take place in the desired part of the connection.

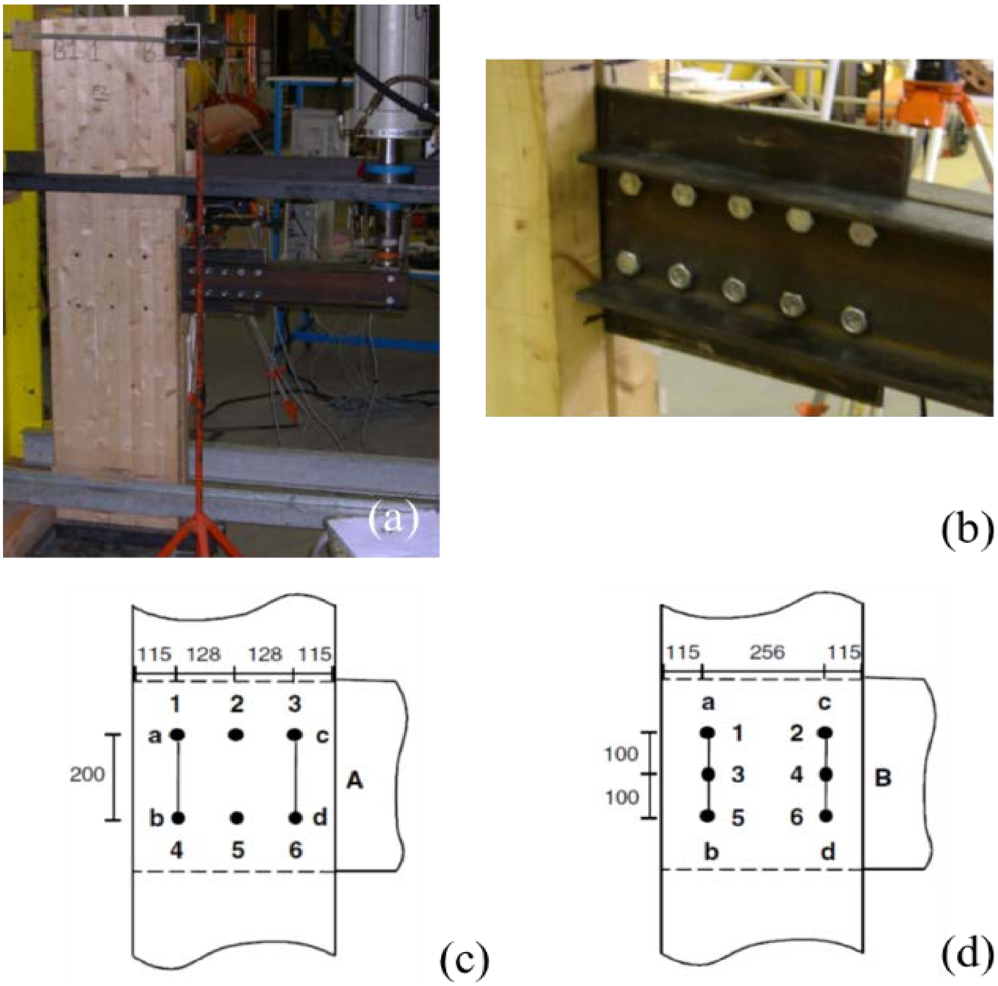


Figure 20. Steel-to-timber moment-resisting (rigid) connection (a) experimental set-up; (b) steel-plate to steel beam bolted connection; (c) dowel configuration A; (d) dowel configuration B.

(Reproduced from Xu et al., 2015, with permission from ASCE.)

The authors did not present a comprehensive discussion of the experimental tests and did not provide conclusions based on them. However, from the test results, it seems reasonable to consider configuration A as a more robust alternative over configuration B as it can withstand larger rotations. Xu et al. (2015) also presented an analytical formulation for predicting the yielding and ultimate moment of the connection, which was conservative in this case. In addition, they showed that the 3D finite element they developed was suitable for replicating with a reasonable degree of accuracy the experimental results.

4.3 Summary

Table 3 presents a summary of the examples of connections reviewed above.

Table 3. Summary of timber-based hybrid connection examples.

Authors	Timber	Other material	Connection type
Bainbridge & Mettem, 1998	Glulam or LVL	Glulam or LVL	Steel plates
Gattesco & Toffolo, 2004	Glulam	Steel plate	Steel rods
Inoue et al., 2004	Sugi timber	RC	Glued-in connectors
Asiz & Smith, 2011	CLT	Steel beam	Screws
Schneider et al., 2014	CLT	Steel beam	Steel tube connector
Hassanieh et al., 2016	LVL	Steel beam	Coach crews and bolts with and without adhesives
Xu et al., 2015	LVL	Steel plate	Dowels

A range of connections that can be used in timber-based hybrid structural applications have been investigated in order to understand how these connections can be used in multi-storey buildings in regions of high seismicity. Certainly not all of the connections discussed would be suitable, but several provided the necessary strength, stiffness and ductility to be considered possible solutions for these applications. These include the CLT to steel connections investigated by Asiz and Smith (2011) and the moment-resisting connections proposed by Xu et al. (2015). This indicates the connections for timber-based hybrid buildings can be designed and, with appropriate testing and analytical analyses, can provide acceptable levels of robustness for use in larger buildings.

5. Practical applications

5.1 Introduction

Just before the end of the 20th century, Banks (1999) addressed the possibility of constructing multi-storey timber structures in New Zealand. He examined the legal maximum number of storeys (and height) for timber structures in the country after the introduction of the New Zealand Building Code in 1992 (Building Industry Authority, 1992), which adopted a performance-based philosophy. This new paradigm described what must be achieved instead of how it must be achieved, relaxing the prescriptive building regulations that were set by the local authorities (Haberecht & Bennett, 1999). Prior to 1992, two codes controlled the height of timber buildings – NZS 3604:1990 *Code of practice for light timber frame buildings not requiring specific design* and NZS 1900.5:1963 *Model building bylaw – Fire resisting construction and means of egress*. Among these, fire considerations were more restrictive, limiting the number of storeys to two plus a mezzanine that could not exceed one-third of the plan area of the floors below it (Tonks, 2004).

Banks (1999) commented that, despite the relaxation of the code provisions, which resulted in a significant increase in the number and size of timber buildings, the tallest structures constructed in New Zealand by the year 1999 were only 5 storeys high. Furthermore, some of these were in fact hybrid structures, as discussed below. Other examples of constructed and designed timber-based hybrid buildings from around the world are also described in this section.

5.2 Constructed buildings

Gulf View Towers (Banks, 1996), shown in Figure 21(a), is a 10-storey residential and car park building situated in Auckland, with the five bottom storeys constructed in RC and the upper five in timber combined with steel elements. The RC part was constructed during the 1960s and had extra capacity to expand by one more RC storey. Alternatively, this existing structure could accommodate five extra timber storeys, as was constructed in the end. The gravity loads in the timber super-structure were resisted by a combination of long-span floor joists and ordinary sawn timber floor joists supported on loadbearing timber-framed walls, loadbearing plywood sheathed shear walls and steel beams. The seismic-resisting system in the longitudinal direction consisted of a series of plywood box shear walls and a moment-resisting frame with plywood box columns and glulam beams. In the transverse direction, on the other hand, lateral loads were resisted by a combination of plywood box type walls and internal plywood walls (Banks, 1996). Tension forces in the shear walls and frames were resisted by steel tie rods anchored at each level using steel plates.

The timber members of the earthquake-resisting system were designed to remain elastic under the design seismic loads required by the New Zealand loading standard at that time – NZS 4203:1992 Volumes 1 & 2 *General structural design and design loadings for buildings*. This standard, like its successor, NZS 1170.5:2004, allowed the designer to place no ductile elements in a structure if the more restrictive loads associated to an elastic design can be resisted by structural elements, which can be quite dangerous (Quintana Gallo, 2014). Banks (1996), opted for an elastic design as above but also provided ductility to the system via the steel tie rods. The latter was justified by considering a ductility-reduced maximum credible earthquake (MCE) loading scenario, equivalent to an elastic ultimate limit state (design) counterpart.

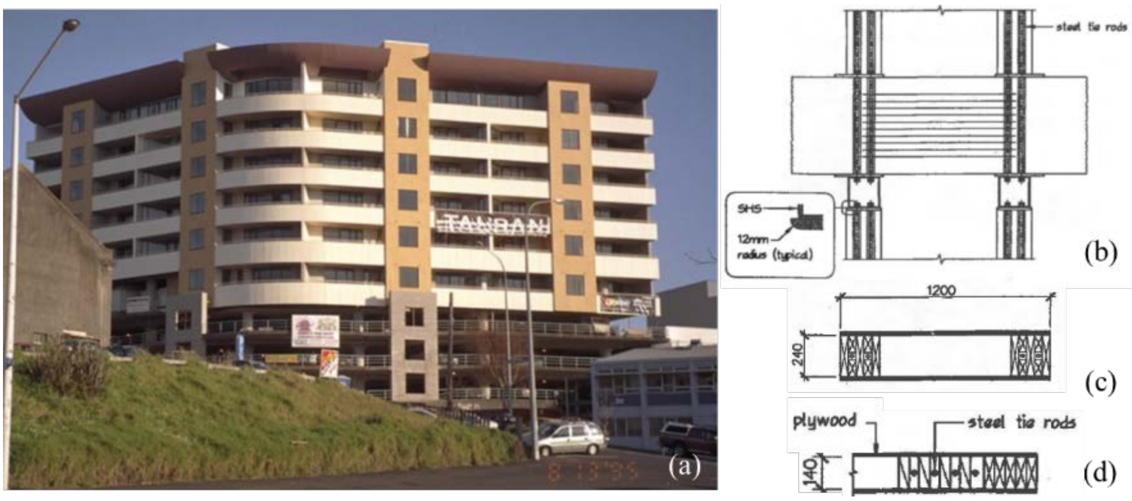


Figure 21. Gulf View Towers hybrid building: (a) overview; (b) beam-column joint details; (c) plywood box column cross-section; (d) plywood box wall cross-section.

(Reproduced from Banks, 1996, 1999, with permission from the author.)

Milburn and Banks (2004) presented a 7-storey residential building constructed in 2004 in Wellington – the region of New Zealand with the most demanding seismic conditions as per code loading requirements (NZS 1170.5:2004).[9] The structure included a reinforced concrete (RC) ground storey with parking places and a predominantly timber super-structure with studio apartments. The conceptual design of this structural system was analogous to that proposed by van de Lindt et al. (2011), with a ground-floor podium made of RC walls and frames instead of braced steel frames. Figure 22 shows the typical plan layout and a rendering of the building on completion.

Figure 22. Hybrid building in Wellington: (a) perspective drawing of the building; (b) super-structure plan layout.

(Reproduced from Milburn & Banks, 2004, with permission from the authors.)

The 6-storey predominantly timber super-structure consisted of an approximately rectangular floor plan (see Figure 22(b)), braced predominantly by plywood- lined walls in each direction, plus a few steel K-braced frames (Milburn & Banks, 2004). The steel

[9] In the recent past, Christchurch, another city in New Zealand, where a large earthquake was not expected, was severely damaged during an earthquake resulting from the rupture of a previously unknown fault. Hence, this standard requirement does not necessarily mean that buildings in Wellington will suffer more severe seismic demands than buildings situated elsewhere, as the data about destructive earthquakes is too small to define such criteria.

K-braces were used to maintain an even distribution of bracing elements across the floor plan and to provide restraint against torsion. The floors at each level of the super-structure consisted of plywood fixed to timber I-joists. The floor system of the ground floor, on the other hand, consisted of an RC slab.

The RC substructure was formed by RC frames and walls, in addition to a few steel bracing elements (Milburn & Banks, 2004). The timber elements consisted of LTF walls sheathed with plywood panels. These elements were not lined up with the vertical elements of the ground floor, producing a vertically irregular structure. However, the designers used RC beams in the ground floor slab to support the timber walls. The reason for this discontinuity was the need for larger open spaces in the carpark.

Vertical irregularity is a downside of this building, as it generates instability in the system. Nevertheless, the stronger resistance and stiffness of the RC members may reduce the detrimental effects of this irregularity, which, in extreme cases, may contribute to the collapse of the building (Quintana Gallo, 2014). Milburn and Banks (2004) also mentioned that the building super-structure was designed with a ductility factor of four (μ = 4), which accounted for the inelastic behaviour that could occur in the nailed connections. This suggests that, under severe earthquake excitations, the dissipation of the seismic energy is taking place in the nails, which means that they will suffer damage, which in turn means that there will be some compromising of the integrity of the timber super-structure.

Koshihara et al. (2009), Isoda et al. (2010) and Koshihara (2013) reported the details of a timber-based hybrid structure constructed in Kanazawa, Japan, in 2004. This building, named M-Bldg, was erected after a modification of the Japanese Building Standard Law in 2000, which permitted the construction of timber structures 4 storeys or higher. This edition of the Japanese code superseded the 1987 restriction that established a maximum of 3 storeys for these structures. Drawings and a picture of the M-Bldg are presented in Figure 23.

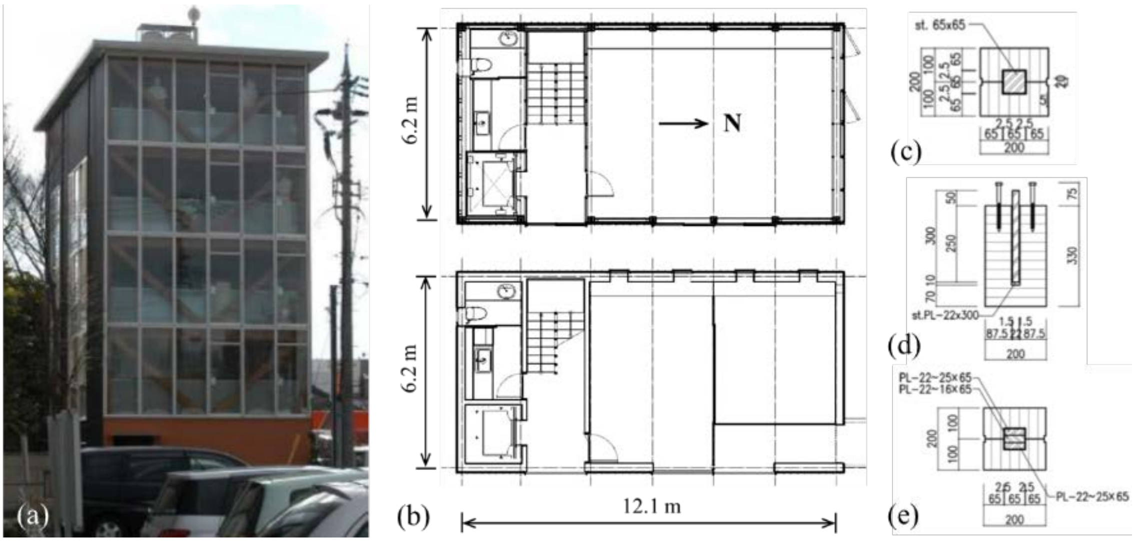


Figure 23. Hybrid building constructed in Kanazawa, Japan: (a) picture from the northwest; (b) plans of substructure (bottom) and super-structure (top); (c) column cross-section; (d) beam cross-section; (e) brace cross-section.

(Reproduced from Koshihara et al., 2009, with permission from the authors.)

The ground and upper four floors of the M Bldg were constructed in RC and timber-steel composite framing and plywood walls, respectively. The composite frames had braces in the short direction of the building only (east and west façades). The braces were placed in an inverted V configuration, as shown in Figure 23(a). The composite columns, beams and braces were formed using steel cores inserted inside glulam members with machined cavities. The cross-sections are presented in Figure 23(c)–(e). The floor system consisted of RC slabs connected to the seismic-resisting system via lag screws and steel plates connected to the beams.

As reported in Koshihara et al. (2009), full-scale replicas of the composite members were tested under monotonic quasi-static loading prior to the construction of the building to check for stability aspects related to buckling of the steel core of columns and braces. The results showed that the steel core could yield and enter the strain hardening range before buckling. However, the surrounding timber failed at rather small deformations.

The M Bldg was the first hybrid structure constructed in Japan. The idea of using braces to stiffen the timber-based structure is sound. However, the use of more robust elements would be desirable. Such robustness can be attained with buckling restrained braces (BRB) as those researched by Blomgren et al. (2016), and, with higher reliability, with traditional steel BRBs.

Koshihara (2013) presented two other examples of hybrid structures constructed in Japan up to 2013 – the Kasukabe Convention Hall and Wood Square Building, shown in Figure 24(a) and (b), respectively. Kasukabe Convention Hall, constructed in 2011 in Kasukabe City, is a 6-storey hybrid building, with the bottom 4 storeys constructed in steel and the top 2 storeys in timber. The timber structure was formed by light timber framing (beam and post) and LVL walls. The Wood Square Building, constructed in 2012 in Koshigaya City, is a 4-storey office building constructed with hybrid steel-timber (glulam) members, like those used in the M Bldg (see Figure 23).

Figure 24. Hybrid buildings in Japan: (a) Kasukabe Convention Hall, Kasukabe; (b) Wood Square Building, Koshigaya City.

(Reproduced from Koshihara, 2013, with permission from Forum-Holzbau.)

Hein (2014) reviewed the state of the practice of hybrid structures in Austria and redefined the possibilities of timber in multi-storey construction, recognising that it is best used in conjunction with other materials. The author reported on collaborative research by Arup and the Austrian developer CREE, amongst others. This research consisted of the development of the concept of a 20-storey building made of prefabricated structural elements, using as much timber as possible. It included the design of the structural, façade, fire, acoustic and building physics systems per the

European, German and Austrian standards. The structural system consisted of one or more glulam timber core walls and frames with timber-concrete-composite (TCC) beams. The floor system comprised TCC waffle slabs with a topping of 120 mm to achieve acoustic requirements.

Based on the concept described above, CREE constructed an 8-storey prototype building – LCT One in Dornbirn, Austria – and a 6-storey 10,000 m^2 office building in Montafon, Austria (referred to here as LCT Two). Pictures of these structures are presented in Figure 25.

Figure 25. Hybrid construction in Austria: (a) 8-storey prototype building (LCT One, Dornbirn); (b) 6-storey office building (LCT Two, Montafon); (c) structural layout of modified prototype to account for seismic actions.

(Reproduced from Hein, 2014, with permission from the author.)

The sizes of the structural elements were slightly reduced to achieve a lower cost. Consequently, they did not meet the acoustic restrictions. However, to improve the acoustic performance, a sound-absorbing raised floor and a self-levelling floor screed were used. The LCT One building included one RC core wall to provide stability to the structural system and a main route of exit in case of fire. Its cost was estimated as 105–110% of a typical office building built in RC.

In 2014, the 20-storey LCT concept building was erected in Germany, a culmination of this research effort on hybrid construction, for low-seismicity areas. The building system was assessed for its application in seismically active regions such as North America (for example, California). The result was the modified prototype shown in Figure 25(c). The most relevant changes included the reduction of the number of storeys to only six, the inclusion of precast concrete slabs and the use of two RC core walls instead of only one (see Figure 25(c)). By doing this, greater lateral stiffness and stronger diaphragm action were ensured. In addition, some dissipation of seismic energy was expected to occur in the bolted steel connections between the RC slabs, which could possibly constitute an unnecessary risk and hence a downside of the design as it may compromise the structural integrity between these units

The main finding of these projects was that it was not optimum to use as much timber as possible in the design of the structure. Reconsidering the use of timber within multi-storey construction, it was concluded that hybrid construction is "the sustainable approach that now forms the basis of design" (Hein, 2014, p. 42). Even though timber is still the material of choice, its weaknesses are recognised as follows:

- Traditional timber connections are relatively weak.
- The thermal and physical mass for room conditioning and control of sound transmission and vibration, respectively, is rather low.

Thus, all the elements of the concept now are aimed at being hybrid, with concrete being a desirable material as it is cheap, dense, easy to install and fireproof. The connection system was reconsidered, this time including screws in plastic tubes and/or fully threaded rods embedded in the concrete slab to fix it into the beams below (see Hein, 2014, for details).

The practical implementations of hybrid structures presented by Hein (2014) represented a very important contribution for the construction of these structures. By evaluating the role of timber within multi-storey construction, the author presented a case with strong grounds for the inclusion of other materials such as concrete into the timber multi-storey construction paradigm, recognising the need for the hybridity of such systems to be feasible. The main downside of the investigation was that these structures were conceived primarily for low-seismicity regions. As such, although there is some progress reported towards this direction, much more research is needed to establish the adequacy of the proposed prototypes in seismic regions. Finally, the idea of dissipating seismic energy in the connections of the structural elements might jeopardise the integrity of the systems during seismic events, as previously discussed.

Fast el al. (2016) presented the structural design of Brock Commons, an 18-storey hybrid building constructed at the University of British Columbia in Vancouver, Canada, as a student residence hall. This project, finished in 2017 at a cost of CA$51.5 million,[10] is the tallest timber-predominant hybrid building in the world, standing 53 metres above ground. The structural system is shown in Figure 26.

[10] Cost as per 2016.

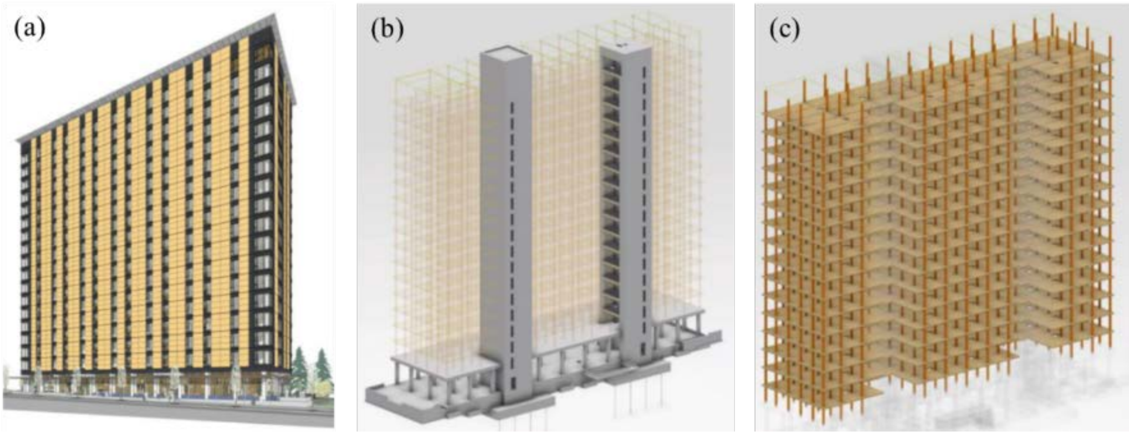


Figure 26. Brock Commons, a timber-based hybrid 18-storey building in Canada: (a) architectural concept; (b) RC first storey (podium) and core walls; (c) timber frames.

(Reproduced from Fast et al., 2016, with permission from the authors. Image credits: (a) Acton Ostry Architects; (b) and (c) CADMakers.)

This hybrid structure includes a single ground-storey RC podium plus a 16-storey super-structure consisting of glulam columns, RC core walls, CLT floors and a single-storey prefabricated steel roof at the top. The use of CLT panels for the floor system of the super-structure permitted the non-inclusion of beams, as these panels can resist bending moments in both principal directions, allowing for an unobstructed service distribution. Some of the CLT panels had a custom layup with outer layers machined using stress-rated spruce laminations to:

- allow for longer spans between columns
- provide further lateral stiffness to the system
- reinforce the region close to the supports (columns) as the rolling shear stresses can control the design in these areas.

The primarily seismic and wind-resisting members of the structure are the two RC core walls. RC was selected for these walls mainly because the testing, time and costs required to obtain regulatory approvals for other materials would have negatively impacted the budget and the duration of the project (Fast et al., 2016). To ensure adequate seismic performance of the floor diaphragms, these members and their connections with the vertical elements were designed using the shear forces that resulted from the walls yielding in flexure.[11] To drag the diaphragm forces to the core walls, steel straps were also included within the connections system (see Fast et al., 2016, for detailed drawings of the connection). The foundation slab was also capacity designed to resist the probable flexural strength of the RC walls, two times the design overturning moment. The column-to-column connections were made of steel and had 1.6 mm thick steel shim plates to accommodate glulam column shortening.

The sequence of construction of the super-structure consisted of two main stages:

- Erection of the RC cores.
- Installation of the timber structural elements and non-structural envelope.

[11] The authors mention the yielding force as a reference for the capacity design of the diaphragms, but in theory, the ultimate strength accounting for over-strength should be used.

The latter stage, in turn, included the following steps:

- Erection of all the columns of one level.
- Installation of the CLT panels.
- Installation of the steel drag plates and perimeter angles to support the curtain walls.
- Installation of the curtain panels in the storey below the active deck (Fast et al., 2016).

The estimated time for the installation of the timber components was 1 week per floor.

The structure under construction presented by Fast et al. (2016) represents the largest and tallest hybrid building in the world constructed in seismically active regions to date. As such it constitutes a milestone and a potential turning point for timber-based hybrid construction.

5.3 Proposed for construction

Banks (1999), after reviewing the state of the practice of multi-storey timber buildings in New Zealand, proposed a structural layout for constructing 14-storey hybrid structures.

He considered two systems:

- Case 1: timber gravity frames and flooring system plus an RC core wall.
- Case 2: steel moment-resisting frames and composite timber long-span floor joists and plywood flooring.

These proposals are shown in **Error! Reference source not found.**.

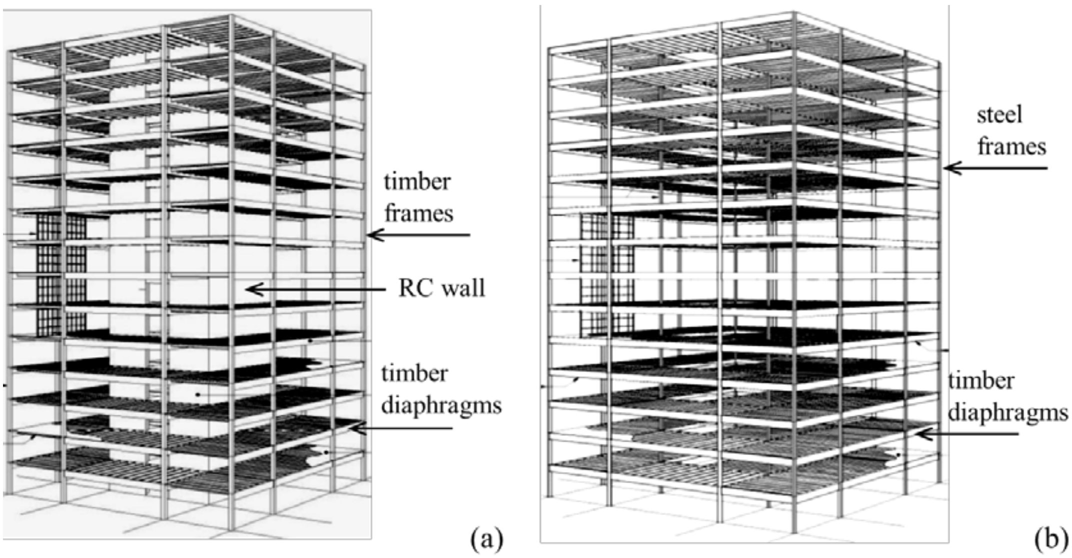


Figure 27. Hybrid systems proposed by Banks (1999) for 14-storey buildings: (a) case 1: timber gravity frames and flooring plus RC core wall; (b) case 2: steel frame plus timber flooring.

(Reproduced from Banks, 1999, with permission from the author.)

As stated by Banks (1999), in case 1, the RC core wall resists all the lateral load whilst the timber frames resist gravity loads only. This argument is true from a force-based perspective but not from a displacement-based point of view. Even if the displacements are rather small because of the large stiffness of the RC wall, the core and frames will both deflect during a seismic event, especially when using a flexible diaphragm. Hence, it must be ensured that the timber elements can accommodate the anticipated local strains they would experience. In case 2, only steel is used in the seismic-resisting structure. As such, it is a hybrid structure, but not a structure with a hybrid seismic-resistant system. It is important to note, though, that this alternative has been followed by other engineers and researchers after 1999. Therefore, Banks' (1999) work was arguably visionary for its time.

John et al. (2012) presented a study where the authors compared the feasibility of constructing a building in Christchurch, New Zealand, after the 22 February 2011 earthquake, using three solutions:

- A hybrid system (HS).
- An all-timber system (AT).
- A conventional system, including RC and structural steel (CS).

The building concept and the location of the site are shown in Figure 28.

These three designs were the result of a selection criteria to produce an optimum hybrid design using what the authors called "the right material for the right application – for a proposed multi-storey commercial development as part of the Christchurch re-build" (John et al., 2012, p. 11). It was required that all the systems must provide damage-resistance and re-centring capacity (high performance) as well as being environmentally sustainable.

The structural elements involved in the design of the buildings were purlins, rafters, structural framing, structural walls and foundation system,[12] each of them having a set of alternatives in varied materials and technologies. The alternatives for the structure included bolted LVL, bolted glulam, bolted steel, precast RC and post-tensioned steel with K-braces for frames and post-tensioned CLT, post-tensioned LVL, post-tensioned precast RC (PRESSS technology), cast-in-place RC and RC masonry for walls. The selection criteria included cost, seismic performance, weight, bulk (volume), acoustic performance, fire resistance, environmental sustainability, speed of construction, durability and neighbourhood impacts.

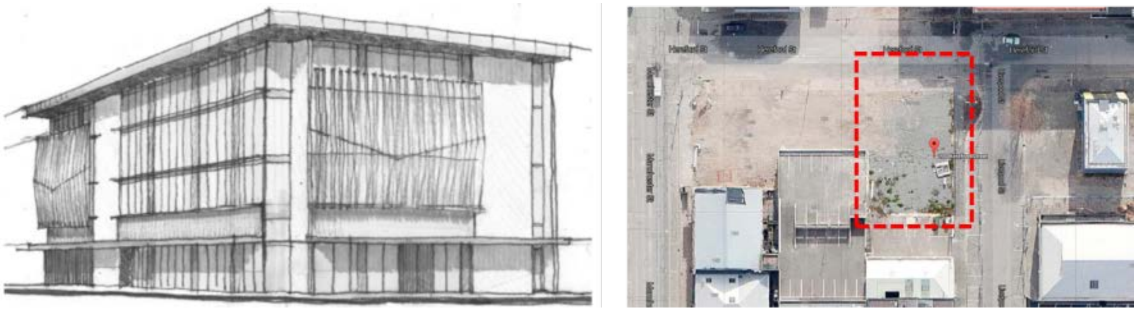


Figure 28. Feasibility of hybrid construction: (a) building concept (b) site location[13].

((a) Reproduced from John et al., 2012, with permission from the authors; (b) Reproduced from Google Maps, copyright Google, MapData Sciences Pty Ltd, PSMA.)

[12] An RC slab was selected for all the systems.
[13] The map shows the site after the deconstruction of the damaged buildings in Christchurch was approximately finished. At the time of writing this report, the site is used as a car park.

The authors concluded, based on their estimations, that the optimum hybrid system would include:

- timber LVL frames
- reinforced concrete post-tensioned rocking walls
- CLT floors (320 mm stress-skin system)
- steel purlins and rafters for the roof.

Hence, the hybrid structure (HS) combined timber, concrete and steel, in that order of volume. The cost of this and the AT (all timber) and CS (concrete steel) structures were NZ$1.8M, NZ$2.4M and NZ$1.6M, approximately,[14] showing that medium-rise multi-storey buildings can be built at a cost comparable to conventional materials (i.e. RC and structural steel). The results are promising for hybrid construction. However, other structural elements such as steel buckling restrained or viscoelastic braces, base isolation, CLT walls, passive and semi-active control devices, amongst others, can be included in future research work.

5.4 Summary

Timber-based hybrid buildings have been constructed in New Zealand and around the world, and several examples have been discussed. Other projects using combinations of timber and other materials have been proposed and used as example projects to show that these types of buildings are not only possible but potentially high performance, sustainable and cost-effective.

Table 4 provides a summary of the practical applications of timber-based hybrid buildings discussed in this section.

Table 4. Summary of practical applications of timber-based hybrid buildings

Authors	Timber	Other system	Location	Status
Banks, 1996	LTF walls; steel-reinforced LTF	RC podium	Auckland, New Zealand	Constructed
Milburn & Banks, 2004	LTF walls	RC podium; steel K-braces	Wellington, New Zealand	Constructed
Koshihara et al., 2009; Isoda et al., 2010; Koshihara, 2013	Glulam frames	RC podium; steel frames	Kanazawa, Japan	Constructed
Koshihara, 2013	LTF walls; LVL walls	RC podium	Kasukabe, Japan	Constructed
Koshihara, 2013	Glulam frames	Steel frames	Koshigaya, Japan	Constructed
Hein, 2014	Glulam frames; glulam walls	TCC floors; RC core walls	Dornbirn, Austria	Constructed
Fast et al., 2016	CLT panels; glulam frames	RC podium; RC core walls	Vancouver, Canada	Constructed
Banks, 1999	Glulam frames	RC core walls	NA	Proposed
Banks, 1999	Timber floors	Steel frames	NA	Proposed
John et al., 2012	LVL frames; CLT floors	RC rocking walls; steel roofing	NA	Proposed

[14] Cost as per 2011.

6. Code provisions and design recommendations

6.1 Introduction

Following the US-Japan 5-year collaborative research project on RC-steel hybrid structures established in 1992 (Goel, 2004) and carried out from April 1993 to March 1998, guidelines for the seismic design of such structures were produced by Nishiyama et al. (2004). These guidelines included two methods:

- The working stress design method, which satisfied the most prevalent seismic provisions of Japan at that time.
- An alternative method suitable in the context of Freeman's capacity spectrum (Freeman, 1978, 2004), permitted by the Japanese standard from 2000 onward.

The concept of the design procedure proposed by Nishiyama et al. (2004) can be taken as a starting point for the development of design guidelines for other hybrid structures. The adoption of Freeman's capacity spectrum design method represented a step forward from a traditional elastic spectral modal analysis, as it directly included the inelasticity of the structure into the calculations. However, as the method included a traditional pushover of the structural model, the drawbacks previously mentioned in section 3 for that type of inelastic method of analysis stand in this case (see Krawinkler & Seneviratna, 1998).

This section presents:

- research that intends to provide values for reduction factors and/or system ductility factors to allow the design of timber-based hybrid structures using a force-based design approach
- research that provides design procedures and/or recommendations for these structures within a displacement-based design philosophy.

In addition, recommendations for the seismic design of all-timber structures are included for the sake of completeness of this report, as several codes require hybrid structures to be designed as if they were entirely made of the most restrictive material. The design recommendations for the seismic design of reinforced concrete-steel hybrid structures are reviewed first.

6.2 Force-based approaches

Ceccotti and Sandhaas (2010) proposed a simplified method for determining the value for the seismic modification factor of timber buildings. This factor corresponded to the reduction factor of the elastic acceleration spectrum of a site (Biot, 1943), which converted it into a design spectrum that accounted for the actual inelastic behaviour of a structure. This method followed the equal displacement or equal energy axioms proposed by Nathan Newmark and collaborators (Veletsos & Newmark, 1960) within a force-based seismic design.

This kind of reduction factor, named R in this report, depends on the materials and the type of structural system used in a specific building. As timber is a relatively new material for multi-storey buildings designed with this method, Ceccotti and Sandhaas (2010) presented a combined testing-modelling approach for determining the R-factor of such structures and others in general. This approach required the performance of monotonic and cyclic quasi-static tests of representative subassemblies to develop and calibrate mathematical models used to conduct NLDA with a set of ground motions, each of them with increasing amplitudes. The variable of control of the amplitude of each earthquake acceleration record, the peak ground acceleration (PGA), was increased until the model reflects a near collapse limit state, obtaining PGA_{NC}. This value, divided by the PGA of the elastic (unreduced) spectrum (PAG_D), yields the proposed formula for calculating R:

$$R = PGA_{NC}/PGA_D \quad (1)$$

The procedure can be summarised in the following steps:

- Construct the model of a building designed with R = 1, for a given PGA_D, calibrating the inelastic properties of the structural members with quasi-static cyclic tests.
- Select a suite of earthquake ground motions covering a wide frequency range.
- For each record, conduct NLDA increasing the PGA (or amplitude of the excitation) until the near-collapse criteria is achieved, obtaining PGA_{NC}.
- Obtain, for each record, a reduction (or performance) factor R with Eq. 1.
- Combine all the results obtained to provide an R as a property of the system under consideration.

Based on the application of this procedure, Ceccotti and Sandhaas (2010) proposed R = 3 for CLT constructions. However, it is important to mention that only eight records were used and that there was one case when R = 4.5 and another when R = 2.5, showing a significant scatter in the results.

Despite the conceptual problems associated with reduction factors and force-based seismic design, the procedure proposed by Ceccotti and Sandhaas (2010) deserves some attention regarding the logic behind it. The amplitude of an elastic acceleration spectrum of a ground motion corresponds to the maximum acceleration (S_a) that an elastic single degree of freedom (SDOF) system can experience when subjected to that ground motion for a given natural period of vibration T_n. The S_a for T_n = 0 corresponds to the PGA of the record. The intention of the reduction factor in a force-based design approach is to provide an estimation of the maximum S_a that an equivalent inelastic SDOF system would experience in reality. As such, the amplitude of the spectrum is reduced, and this includes the PGA. In the procedure proposed by Ceccotti and Sandhaas (2010), the NLDA are conducted using a record compatible with the elastic spectrum first and then increasing the magnitude of its PGA, which amplifies the whole amplitude of the motion. Hence, the numerical model is being increasingly subjected to a record with larger S_a than those of the elastic case of reference. Consequently, the nominal collapse of the structure is achieved with a compatible spectrum, which, when reduced by R, yields the elastic spectrum of reference, contradicting the hypothesis of a force-based approach, which stated that the amplitude of the design spectrum is smaller than the elastic counterpart. On a practical side, the acceptance of a given R factor enables the use of this methodology for a method already approved for being used in the practice worldwide, facilitating the construction of timber structures in seismic regions.

Dickof et al. (2014) evaluated the ductility and over-strength of hybrid CLT-infilled steel frames, as discussed in section 3 of this report, using an equivalent static force-based design approach. This method used the S_a associated to the first elastic mode of vibration of a structure to calculate an equivalent base shear. To obtain the base shear, this S_a, obtained with the National Building Code of Canada seismic standard, was multiplied by the seismic weight of the structure, a higher-mode amplification and an importance factor, and it was divided by the over-strength factor R_o and a ductility-related reduction factor, R_d.

The ductility of the system under investigation was calculated with pushover analyses (section 3). As the determination of a nominal yielding point for timber (and timber-based) structures can be a difficult task due to the absence of a classical yielding point as in RC (Ceccotti & Sandhaas, 2010), Dickof et al. (2014) used the equal-energy method to define this point in their pushover curves and compute the ductility factor (μ). The R_d factor was taken as being equal to μ, as first proposed by Veletsos and Newmark (1960) (see also Riddell & Newmark, 1979; Newmark & Hall, 1982), which made use of the equal displacement axiom. The results showed that the lowest 97th percentile ductility of an infilled configuration of any of the frames investigated was 2.5, which, per Dickof et al. (2014), appears to be warranted from the preliminary study. The system over-strength was calculated as the ratio of the maximum base shear obtained with the pushover analysis and the design base shear in accordance using FEMA P695 (Applied Technology Council, 2009). The results showed similar values in all cases, which led the authors to suggest the value R_o = 1.25 for the new system.

Regardless of the adequacy or not of the force-based design approach adopted by the authors, the main limitation of the work presented by Dickof et al. (2014) is related to the estimation of the ductility capacity of the system. As mentioned by Ceccotti and Sandhaas (2010), the yielding displacement (Δ_y) of timber structures is difficult to determine due to the shape of the force-displacement plots. As μ is directly proportional to Δ_y, any error in the determination of the latter will be reflected in the former. Furthermore, if it is not possible to determine Δ_y, it is not possible to determine μ. With regards to the over-strength factor, the proposition of a unique value can be useful for design purposes and to have a general idea, but as the value is determined using pushover analyses of relatively simple configurations, it is possible that the same value is not adequate for more complex building configurations. Moreover, as the method of analysis is relatively simple, it could be argued that it can also be used to determine R_o on a case-by-case basis. Finally, experimental data should be obtained to support the findings.

Zhang et al. (2016) proposed a ductility factor (R_d) for the seismic design of the FFTT system (previously reviewed in this report) in the context of a force-based approach. The authors discussed that there is a need for those factors, as in North America, they are not included in the loading standards such as the National Building Code of Canada). To establish a number, Zhang et al. (2016) conducted a series of two and three-dimensional NLDA on a numerical model of option 1 of the FFTT system (Figure 10(a)) with 12 storeys. For details of these models, refer to Fairhurst et al. (2014) and Zhang et al. (2016). Based on the results of these analyses, the authors concluded that R_d = 5 can be preliminarily recommended for the FFTT system. The benefit of defining this factor is related to the constructability of the system, as it fits into the accepted methods of analysis and design. However, the same problems with a force-based approach remain.

Pozza et al. (2016) also proposed a force-based design for their hybrid system described previously in this report. Using NLDA, they calculated a reduction factor (or performance factor, *q*, as per BS EN 1998-1:2004 *Eurocode 8. Design of structures for earthquake resistance: General rules, seismic actions and rules for buildings*) to reduce the elastic spectrum. Using a model implemented in the computer program OpenSees (McKenna et al., 2000) and the procedure proposed by Ceccotti and Sandhaas (2010), they proposed to design their hybrid system with R_d = 4.

Tesfamariam et al. (2015) presented a report with a force-based design guideline for CLT-infilled steel frames, examined previously (Dickof et al., 2014). This guideline included all the work of the researchers involved in the project, including experimental results on the connecting brackets between CLT and steel, numerical modelling, parametric studies on these models, calculation of over-strength and ductility factors using pushover analyses and NLDA to evaluate the adequacy of the proposed modification factors. This design guideline presented important aspects that should be included in such a document. However, a significant drawback is that there was not a single experiment conducted on the proposed system that could support the conclusions reached numerically. Hence, further work that includes quasi-static and or shake table testing is needed to support or refute those findings.

6.3 Displacement-based approaches

Bezabeh et al. (2016a) presented an iterative direct displacement-based design (DDBD) procedure for the hybrid system proposed by Dickof et al. (2014), which included CLT-infilled steel moment-resisting frames (Figure 13). This method, initially proposed by Priestley (1998) and published in a comprehensive book in 2007 (Priestley et al., 2007), used the concept of a substitute structure (Shibata & Sozen, 1974). This substitute structure was an equivalent single-degree of freedom (SDOF) system assumed to represent the real inelastic multi-degree of freedom (MDOF) system with equivalent elastic properties. These properties included effective mass (m_{eff}), stiffness (k_{eff}), height (h_{eff}) and period of vibration (T_{eff}), an equivalent design displacement (Δ_D) and ductility factor (μ) at the effective height and equivalent viscous damping (ξ_{eq}) that included velocity-related and hysteretic-related damping.

The specific procedure proposed by Bezabeh et al. (2016a) can be divided into the following steps:

1. Assume values for the properties of the system (size of the gap between steel and CLT; connection spacing; CLT panel thickness and strength; initial and post-yield stiffness of the steel members).
2. Assign a strength proportion between CLT and steel members.
3. Define a design displacement profile for the structure.
4. Define Δ_D, h_{eff}, m_{eff}.
5. Define μ at h_{eff}, using a weighted average of the ductility associated to the CLT and steel.
6. Obtain ξ_{eq} using μ and calibrated formulas (Bezabeh et al., 2016b, in this case, which is based on Dwairi & Kowalsky, 2007, and Grant et al., 2005).
7. Obtain T_{eff} using a design (reduced) spectrum (Bommer & Mendis, 2005).
8. Calculate k_{eff} with T_{eff} and m_{eff}.
9. Obtain the design base shear for the structure, V_D.
10. Distribute the forces in the structure using a rational criterion, different from a traditional force-based approach (see Priestley, 2003, for example).
11. Calculate the properties of the system and compare to (1).
12. Repeat all the steps until (11) is acceptably close enough to (1).

Bezabeh et al. (2016a) also presented a design example of their hybrid system (Dickof et al., 2014). They validated[15] the design using NLDA. However, they modified all the ground motions used as inputs to match their design spectrum. As such, it would be hard to expect that the results would not confirm their design.

The procedure proposed by Bezabeh et al. (2016a) made use of DDBD, which is a relatively well accepted method among the engineering research community. However, there are some aspects of this approach that deserve close attention. Among these, it is important to bear in mind that this method uses a reduction factor, named $R(\xi_{eq})$, a function of ξ_{eq} (Bommer & Mendis, 2005), which presents similar philosophical problems as the reduction factor used in force-based design. Despite this, the proposal of Bezabeh et al. (2016a) represents a step forward in seismic design as it places focus on displacements rather than on forces, which is a more rational approach for the seismic design of structures in general.

6.4 Summary

This section included discussions on different methods for determining the design actions on hybrid buildings using both force and displacement-based methods. Many were based on buildings and concepts used by authors in previous sections to further understand how different timber-based hybrid systems can be designed using existing building codes and methods. While there are shortcomings to many of the proposed methods, there is enough guidance to provide a framework for designing these buildings within existing design paradigms.

Table 5 presents a summary of the design approaches included in this review.

Table 5. Summary of proposed design methods

Authors	Approach	Proposition
Ceccotti & Sandhaas, 2010	Force-based	Methodology for calculating R factors
Dickof et al., 2014	Force-based	Ductility factor; over-strength factor
Zhang et al., 2016	Force-based	Ductility factor
Pozza et al., 2016	Force-based	R factor
Tesfamariam et al., 2015	Force-based	Design guide
Bezabeh et al., 2016a, 2016b	Displacement-based	Design methodology

[15] Per Popper (1963), ‘validation’ does not correspond in empirical science, as one requisite of any empirical theory is refutability. As such, there should always exist the possibility of that theory to be refuted with a counterexample.

7. Summary and recommendations

Reinforced concrete and steel have traditionally been viewed as the optimal structural materials for large multi-storey buildings. Timber is being increasingly utilised in buildings over 3 storeys around the world, and designers and manufacturers are making efforts to meet this need through designs, products and systems that incorporate wood both for structural and non-structural applications. It has been noted that the most efficient use of resources results from combining timber with other materials so that the beneficial properties of each material can be used in ways that capitalise on their inherent properties to the most advantage.

This report discusses research literature on timber-based hybrid buildings and in particular how these buildings resist seismic actions. There is enough research presented to conclude that timber-based hybrid buildings are becoming increasingly popular and require continued research to ensure that these buildings are seismically robust and resilient while also being low damage, cost-effective and sustainable. Experimental testing and a variety of numerical analysis methods have been used to validate some of the concepts suggested for these types of buildings and the connections between timber and other materials. Examples of existing timber-based hybrid buildings have been presented as well as building concepts that were developed to investigate the feasibility of these buildings. Design methods including force-based and displacement-based applications were discussed in relation to timber-based hybrid buildings in terms of what has been tried and where there is still some work to be done.

While it would be ideal to conduct full-scale shake table tests of buildings to verify their performance, this is often not a feasible proposition due to the cost, time and effort required. Additionally, there are buildings throughout the world that have proven to be effective in resisting earthquakes that were designed using methods that continue to be implemented and are trusted by designers as conservative enough to provide life safety and in some cases low-damage buildings. Numerical analysis methods should continue to be used to develop timber-based hybrid designs with experimental testing used to validate these designs wherever possible. Research should be continued in this area to further refine models and develop elements for use in numerical models that can be used for a wide range of applications rather than only specific structural cases.

One issue for consideration would also be the availability of generic connectors that can be purchased for these applications and that have been tested and possibly certified as providing the necessary attributes that building designers require. This could also be extrapolated to include standard connections and details, but this would require significant attention to application due to the wide range of buildings and architecture that are possible for different situations.

Design guidance is needed to assist designers of these buildings so that efficient structures can be developed incorporating timber with other materials. Strength, deformation and stiffness compatibility must be considered, and while methods exist that can be applied to these buildings, it would be ideal to have more specific guidance on how to address these issues. This is particularly relevant for regions that require seismic resistance as part of the structural design. There is also a need to consider how the interaction of timber and other materials could create unanticipated effects to the acoustic or fire performance of a structure and how these could be managed.

In summary, this report has identified the following research topics that are recommended for future investigations on timber-based hybrid buildings:

- Specific recommendations for practical methods of dissipating seismic energy that is not focused where connections are made between timber and other materials, such as buckling restrained braces.
- Investigations that include seismic resistance within the design of timber-based hybrid buildings and are not intended only for non-seismic regions.
- Further development of design factors that apply specifically to differently configured timber-based hybrid buildings.
- Verification of overall building performance using test methods that include dynamic response such as shake table testing.
- Additional design guidance so that code-compliant designs can be developed and consented with confidence.

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