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Seismic protection technologies for timber structures: a review

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Abstract

Timber structures traditionally provided satisfactory seismic performance due to multiple known features. However, the consequences of the last major earthquakes have clearly proofed that seismic timber design must further improve. In addition, nowadays timber structures target taller heights and so they face much larger seismic demands. All this together has made seismic protection technologies (SPTs) to emerge as a hotspot in timber engineering research, devoting more than 80 publications only in the last decade. All types of SPTs share the common principle that, rather than increase the lateral resistance of a structure, they are focused on reducing the seismic demands and such reduction has been reported as large as 90% and above. Although many distinct devices and techniques are intended to this end, SPTs applied to timber structures may be grouped into supplemental damping, seismic isolation, and rocking systems. Apart from the copious scientific production in the field, knowledge has been published in very distinct niches, which makes a linkage of state-of-the-art very difficult, as well as an analysis of current challenges and limitations. This review attempts to provide so after explaining first the basic principles of these technologies so that they are comprehensible for a timber engineer or researcher not necessarily familiar with all structural dynamics' underlying concepts. An outlook for future research trends is expected towards cost-effectiveness, rate-effects, engagement of devices, and design guidelines which may expand these technologies bringing timber structures into higher levels of seismic performance.

1 Introduction

Timber structures are widely acknowledged for their capacity to withstand very intense earthquakes without collapsing, which has been proved by the very small percentage of causalities registered in timber structures, only 0.5% during the last major earthquakes (Rainer and Karacabeyli 1999). Such inherent seismic performance is mostly attributed to its lightweight, specific strength and stiffness, structural redundancy, elastic deformation capacity and ductility of connections. Nonetheless, timber construction is not exempt from suffering significant damage as demonstrated by recent earthquakes, especially for houses that often lack structural design.

For instance, in the 1994 Northridge earthquake (M_W = 6.7), most light-framed wood houses were built following prescriptive guidelines rather than a seismic design code. Consequently, half of the \$40 billion on property damage was reported in light-framed wood houses, resulting in almost 48,000 of them declared uninhabitable (Symans et al. 2002a). Besides, some taller light-framed buildings (3- to 4-stories) showed a soft-story failure mechanism causing 24 out of the 25 casualties reported during this earthquake (Symans et al. 2002a). Similarly, the majority of causalities in the 1995 Kobe earthquake (M_W = 6.9) was registered in one- and two-story wooden houses built with the traditional Japanese post-and-beam system. Most of these houses were built after World War II when the lack of policies for house restoration and the construction boom affected the quality of houses (Prion and Filiatrault 1996). Vast economic losses also related to low rise lightweight timber construction during the aftershock of the Christchurch 2011 earthquake (M_W = 6.7). The scientific community has therefore recognized that the seismic behavior of timber houses must improve.

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Besides single-family housing, current developments in construction target taller timber buildings (Pei et al. 2016)—that experience much larger seismic forces—and may use different structural systems, connections, and products, such as massive timber systems whose ductility largely depends on few sparse connections (Karacabeyli and Lum 2014). These systems are recent and suffered virtually no exposure to severe earthquakes except those simulated in laboratory conditions, limiting the empirical evaluation of their performance. Therefore, these new systems must be thoroughly studied in order to propose appropriate seismic design guidelines.

The core of traditional seismic design of timber buildings comprises the definition of an adequate seismic force resisting system (SFRS) that provides most of the lateral capacity by including one or several of the following lateral resisting systems: (i) moment resisting framing; (ii) bracing; (iii) light-frame shear walls; (iv) massive timber shear walls; (v) and structural cores. However, a complementary approach of the above is given through the most recent concept of seismic protection technologies (SPTs), which entails a series of devices being the objective to reduce seismic demands on structural components rather than increasing the lateral capacity of the building. Specifically, seismic protection is typically implemented through the inclusion of isolators and dampers, such that structural deformations and internal forces in members are reduced. This makes possible to achieve much higher standards of seismic performance - even damage-free—after design level earthquakes as well as facilitating retrofitting measures and thus improving resilience.

Given that seismic protection has been regarded as a sophisticated design approach, most applications involved mid- to high-rise multi-story concrete and steel buildings and implementations in timber structures were rather modest during the last decades of the twentieth century. Actually, the first SPTs in timber essentially focused on the direct application of concrete and steel SPTs, as reviewed by Symans et al. (2002b). Early research showed that utilization of SPT in timber was feasible and beneficial even at low-rises, reducing drifts and force demands besides being a little intrusive method to retrofit vulnerable historical buildings. In the last two decades however, the increase in the demand for multi-story timber buildings, more restrictive structural requirements and the development of cost-effective devices have made SPTs to rapidly increase, becoming a hotspot in timber engineering research. Most recent advances tailored and developed innovative solutions under consideration of the peculiar features of timber buildings. Still, in most cases, the research in this field has fallen very close to implementation, and publications can be found scattered into very distinct sources, which makes a linkage of the state of the art very difficult, as well as realizing current challenges. This review provides an overview of the state-of-the-art of SPTs in timber buildings, focusing on the description, developments, potential, limitations, and challenges for future development. The paper is organized by first introducing the SPT fundamentals, then describing those SPTs that have been implemented in timber buildings so far, and finally an overview of the potential, limitations, and challenges for future outlook.

2 Principles of seismic protection and dynamic behavior of unprotected timber structures

The principles underlying SPTs can be understood in terms of the energy balance of a system subjected to seismic excitation. The ground motion implies an energy input into the structural system being transformed into four different energy outputs that keep in balance with the input, see Eq. 1 (Christopoulos et al. 2006): (i) the kinetic energy, stored as motion; (ii) the potential energy, stored as elastic deformation; (iii) the energy dissipated through inherent damping due to material frictions, heat and similar losses; (iv) and the energy dissipated through structural and non-structural damage, which, in a properly designed timber structure mostly comprises the hysteretic (ductile) behavior of the connections (Eq. 2).

$$E_{\text{input}} = E_{\text{output}} \tag{1}$$

$$E_{\text{input}} = E_{\text{kinetic}} + E_{\text{potential}} + E_{\text{damping}} + E_{\text{damage}} \tag{2}$$

Energy balance mechanics is exemplified in Fig. 1a. Notice that input and dissipated energies are both cumulative as they always increase; however, kinetic and potential energies oppositely fluctuate because these two mechanisms only store the input of energy until it is released. The energy that can be stored as potential energy has a maximum limit given by the elastic deformation capacity of the structural components, upon of which either inelastic deformations in connections or structural damage in timber members—very often brittle due to combined bending, shear or tension failures—occur being thus dissipated as damage.

The energy balance may be seen through a rain-flow analogy as illustrated in Fig. 1b (Christopoulos et al. 2006). The input energy may be regarded as the rainwater collected by the structure through a plumbing system (the structural mass and rigidity) storing such energy flow in kinetic and potential pails. Those pails continuously exchange water with each other when full via pump system (mechanical energy conservation), unless a certain (elastic) limit is exceeded in the potential pail, in which case the overflow leaks into a damage pail (the inelastic behavior of connections or brittle damage of members). In the exchange of potential and kinetic energy, a small flow is also lost due to inherent damping

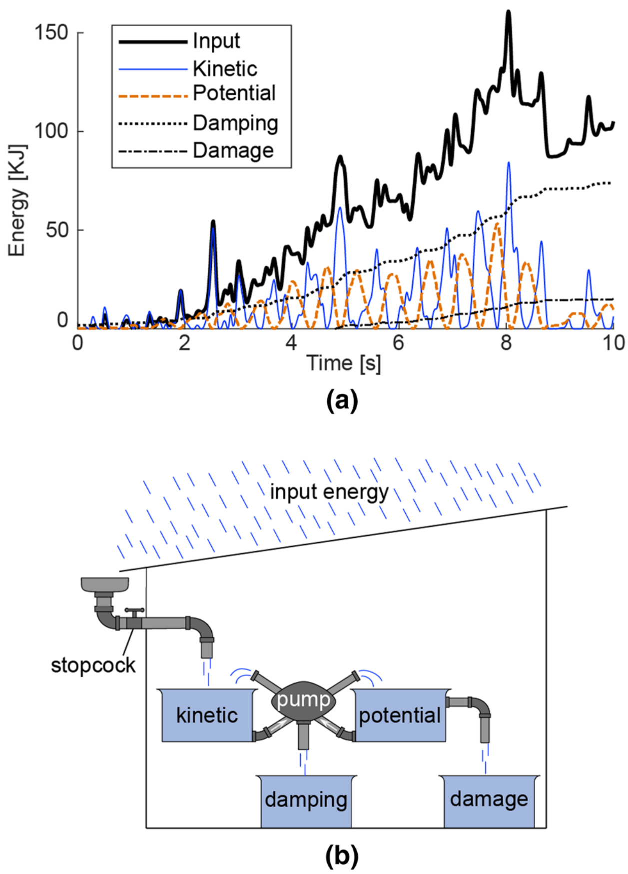

Fig. 1 **a** Typical energy plot during a ground motion. **b** Rain-flow analogy for the seismic energy flow after Christopoulos et al. (2006)

(internal friction, heat, etc.), commonly referred to as linear viscous damping.

It is recognized that the potential energy capacity will always be exceeded upon design earthquakes. Thus, the ability of an unprotected timber structure to prevent collapse entirely relies in that the damage must be dissipated through the hysteresis of the connections and not through brittle damage of members or elastically designed joints, which turns to oppositely compare to the traditional seismic design approach of common steel structures (Karacabeyli and Lum 2014). Therefore, the basis of the timber seismic design approach consists of guarantying that the inelastic deformation capacity of connections (ductility) is fully exploited before brittle failure capacities in members are reached, and this is referred to as the capacity design approach. Because in static design, both the members and the connections are designed conservatively, it is crucial to know the true 95 percentile capacity of connections so as to overdesign timber members ensuring that brittle failure is always subsequent to ductile one. The difference between the analytical 5% and true 95% capacity of ductile connections is termed as the overstrength (γ_{Rd}) needed in the brittle components (Jorissen and Fragiacomo 2011; Ottenhaus et al. 2018). Although inelastic connections allow for energy dissipation, they typically suffer severe capacity drops (strength and stiffness degradation) that decrease the energy dissipation being this reflected as the pinching of the hysteretic curve. Thus, seismic forces need to be increasingly withstood by the remaining structure, which makes it very difficult to prevent from failing, especially after repeated earthquakes, and so seismic protection becomes relevant. SPTs aims at preventing the input energy from becoming potential energy, thus reducing damage risk significantly. In order to accomplish this, the kinetic and damping energy terms must increase or the input energy decrease.

In conventional unprotected light-framed timber systems, the seismic input is collected by the horizontal diaphragms and transferred to the walls. Then the walls resist most of the lateral force through the bracing action of the sheathing, as the framing connections have very little rotational stiffness. Actually, the overall deformation and stiffness of a light-framed shear wall is commonly seen as a series springs system comprising the flexural deformation of the chords (end studs), the uplift deformation of the anchorage (mostly a hold-down), the shear deformation of the sheathing and the slip of the sheathing-to-frame nailing (ANSI/AWC 2015). The damage and ductility is mostly concentrated in the latter. Thus, sheathing-to-framing nailing casts the weak link of the assembly. The seismic performance of this system has been widely studied, with the first tests in the 1970s (Yancey and Somes 1973; Yokel et al. 1973). Contemporary shaking table tests have evidenced the large deformation capacity of the system (Ventura et al. 2002) which behaves as a cantilever during severe accelerations thanks to the restriction provided by hold-down elements (Tomasi et al. 2015a). Most of the tests have shown satisfactory performance with modest levels of damage, often related to sheathing and finishes (Filiatrault et al. 2002; van de Lindt et al. 2010b) or even no noticeable damage after severe earthquake simulations (Tomasi et al. 2015a). However, other tests have instrumentally captured large stiffness and strength degradation, despite the lack of visible damage (Tomasi et al. 2015b; Casagrande et al. 2016). Extensive testing programs evidenced that structural damage of light-frame timber walls consistently relates to inter-story drift, showing inelastic damage onset about 5‰ and large structural damage (collapse) in ranges about 7–8% (Ventura et al. 2002). Due to the high ductility of the system, theoretical elastic seismic forces are commonly reduced by approximately 5–6 in practical design, while the actual reduction is considerably smaller due to the high overstrength of the system: 2.5 according to ASCE 7–10 (2010). Several researchers have highlighted the significant contribution of wall finishes such as gypsum wallboards (GWB) and stucco to increase the stiffness and strength of real walls in comparison to the typical systems tested in laboratories (Filiatrault et al. 2002, 2010; Christovasilis et al. 2009). GWB have been used in the past decades as an alternative to wood structural panels (WSP) such as oriented strand boards (OSB) and plywood (McMullin and Merrick 2002). However, GWB typically provide much smaller capacities and ductility than WSP (Grossi et al. 2015) except when small capacity nailing such as staples are used, in which case the capacity of GWB may compare to that of WSP (Seim et al. 2016). Thin reinforced concrete slabs have also been tested as sheathing, showing feasibility for seismic prone areas (Pozza et al. 2016a).

In recent years, unprotected massive timber systems consisting of glued layers of wood such as laminated veneer lumber (LVL) and cross-laminated timber (CLT) have emerged as an alternative to light-framed systems. These engineered timber products can be used to fabricate beams and columns; however, in seismic prone areas, most applications related to shear walls, given its large in-plane lateral strength and stiffness. In fact, during ground motions, massive timber walls behave mostly as rigid bodies (Popovski et al. 2010) whose typical failure modes include rocking for slender walls and sliding for the wider ones (Izzi et al. 2018b). In the event that consecutive walls are edge-joined, sliding deformation may govern for slender walls as well, depending on the relative stiffness between edge and base connections (Tamagnone and Fragiacomo 2018). As for the previous LFRS, massive walls' deformation and stiffness is regarded as a series springs' system comprising shear deformation of the panel, uplift (rocking) deformation, base sliding and, in some models, also the panel bending is accounted for (Brandner et al. 2018). The weak link, and thus ductility, is mostly provided by the connections that prevent walls from rocking or sliding, i.e. hold-downs, edge joints and angle brackets. Thus, the damage is concentrated in smaller area in comparison to light-framed shear walls; therefore, the ductility of this system is normally less than half of light-framed walls, so that elastic forces are commonly reduced by only 2–3 in practice (Tannert et al. 2018). It is however worth mentioning that metal connectors of CLT have shown significant energy dissipation and ductility capacities (Schneider et al. 2014; Gavric et al. 2015a) mostly due to the reinforcing action of the cross-layers (Brandner et al. 2018). Connectors can also be located at the wall-to-floor interface of vertically segmented multistory walls; however, most of the deformation is concentrated in the wall-to-foundation connection (Popovski and Karacabeyli 2012). Additional energy dissipation is provided by the friction at the base of the wall (Izzi et al. 2018b) although it is often conservatively omitted. The use of edge joints to connect panels in a row enhances the ductility and capacity to dissipate energy (Hossain et al. 2016; Izzi et al. 2018a). The performance of these joints is highly influenced by compliance of fastener edge distance requirements (Gavric et al. 2015b) and the strength of fasteners (Pozza et al. 2016b). Vertical edge joints with screwed metal connectors linking walls with spandrels have shown to be effective in reducing stiffness and increasing damping (Yasumura et al. 2016), as well as corner-to-corner connectors (Polastri et al. 2018); this later also suggested to be an alternative to hold-downs and angle brackets. In general, dynamic tests of CLT structures have shown an adequate seismic performance (Ceccotti et al. 2013; Flatscher and Schickhofer 2015). However, efforts regarding research, marketing and regulations are still required in order to increase the implementation of the system to tall buildings in seismic prone areas (Pei et al. 2016). Although not commonly included in calculations, the irreversible crushing of the wood while rocking may also significantly contribute to overall deformation and dissipation—especially if the slab beneath the wall is also made from timber—up to the extent that hysteresis curve may change significantly with the connections dissipating much less energy (Hummel 2017). Very distinct hysteretic behavior is also observed with large gravitational loads, such that sliding tends to govern deformation even for slender walls, and lateral capacity may increase by 40% or more (Hummel 2017). Despite all previous influencing parameters, again for this LFRS, drift strongly relates to damage with yielding about 2–5‰, and collapse about 2% (Hummel 2017).

As commented above, the connections in both, massive and especially light timber systems provide significant energy dissipation capacity. Some authors argued that this capacity is enough to guarantee a satisfactory seismic performance of conventional (i.e., unprotected) timber structures when they are properly designed, as already evidenced in some shaking table tests. Therefore, in order to justify the implementation of SPTs in timber structures, these technologies should reduce demands before non-linear deformation occurs in the connections, which is in principle technically feasible as yielding drifts in timber walls compare to half of the ultimate drifts of concrete. Hence, SPTs cannot only improve the immediate performance for a given earthquake in comparison to unprotected timber, but also resilience, because inelastic damage of walls is mostly not reversible. Additional motivation for SPTs are the protection or retrofit of vulnerable historical buildings (Reed and Kircher 1986; Mualla and Belev 2017) and the reduction of large accelerations reported in shaking table tests of tall timber structures, which have been deemed as a potential source of non-structural damage (van de Lindt et al. 2010a; Ceccotti et al. 2013).

3 Types of seismic protection

Many types of SPTs and corresponding devices have been proposed to date; however, this review is only limited to those that have been applied to timber engineering so far, in addition, other not yet used devices are also described in the outlook section. The applied devices entail three major categories: supplemental damping, base isolation, and rocking systems.

3.1 Supplemental damping

Inherent damping occurs in both elastic and inelastic structures due to internal frictions and similar losses. Such dissipation is typically expressed through the equivalent viscous damping ratio (ξ), a relative measure of the actual damping of a structure with respect to the critical damping, i.e., the damping required to stop the motion of a system in one vibration cycle. As a phenomenon related to energy dissipation, the characterization of damping, in general, is complex and carries a substantial uncertainty regarding the appropriate model and numerical value to use, given the dispersion reported in the literature (Jayamon et al. 2018). Inherent linear viscous damping coefficient is typically taken as 2–3% when hysteretic damping is considered apart (Priestley et al. 2007). On the other hand, equivalent viscous damping from structural systems depends very much on the number and hysteretic damping characteristics of connections as well as their degree of yielding: typically it is given in ranges from 10 up to 20% (Priestley et al. 2007; Chopra 2016) and even 30% for some structures (Porcu 2017). The total energy dissipated during a cycle of vibration due to inherent damping plus inelastic behavior (if present) is given by the area under a force–displacement (or moment-rotation) hysteretic plot, like that shown in Fig. 2a. The seismic protection through supplemental damping aims at decreasing the structural (potential energy) demands by increasing inherent damping dissipation through the addition of supplemental devices called dampers. These devices can be activated by displacement, velocity or motion increasing the equivalent viscous damping. For equivalent displacements, supplemental damping typically increases the lateral stiffness, thus enlarging forces and shortening fundamental period, but also dissipating much more energy, see Fig. 2a. The increase in damping not only reflects in reducing potential but also kinetic energy, thus reducing both the acceleration and displacement demands. While period shortening usually implies larger acceleration, it leads to smaller displacements so in general supplemental damping turns quite effective in reducing drifts, see Fig. 2b, c. This is especially beneficial for timber structures, as structural damage shows much larger correlations with inter-story drifts than structural forces (Priestley et al. 2007).

Displacement-activated dampers dissipate energy as a function of the relative displacement between the device ends. This category includes metallic dampers, i.e., pieces of metal (plates or bars) intended to yield during ground-motions (Fig. 3a), hence, dissipating energy through plastic deformation. Energy can also be lost via friction during

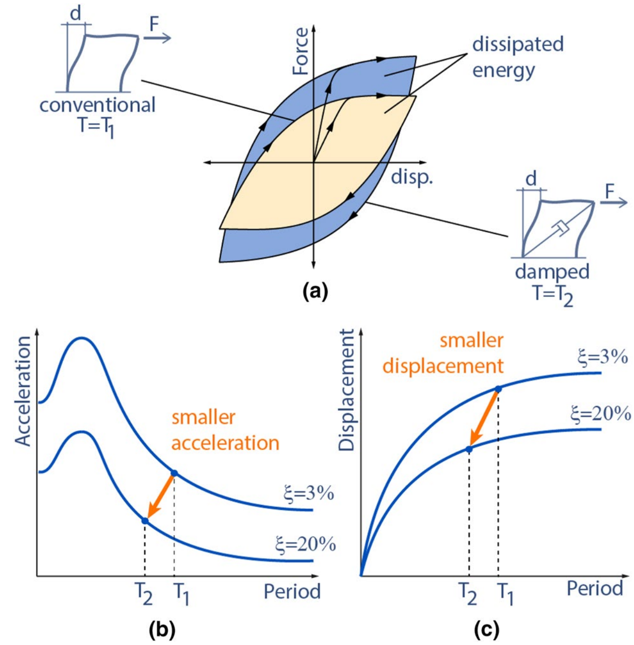

Fig. 2 **a** Idealized force–displacement hysteretic behavior for conventional and damped system. Effect of the larger damping and stiffness in the structural response of **b** acceleration and **c** displacement

sliding which casts the basis of friction dampers consisting of several tightened plates (Fig. 3b). Unlike the hardening driven hysteretic curve of plastic dampers, frictional curves are typically rectangular as they are mostly governed by frictional onset force or moment, see Fig. 3b.

Velocity-activated dampers dissipate energy as a function of the relative velocities between the device ends. This category includes fluid viscous dampers, consisting of a piston and a cylinder whose internal chambers are filled with a viscous fluid and connected with orifices (Fig. 3c). When the ends of the device move, the fluid flows from one chamber to another, dissipating energy in the process. Viscoelastic dampers consist of two surfaces connected with a viscoelastic material (Fig. 3d), i.e., an elastic material showing significant dissipation by viscous deformation, and therefore rate dependent. Viscoelastic dampers have been widely studied, but implementations in civil engineering are rather scarce. Typical elliptical force–displacement plots of viscous and viscoelastic dampers are shown in Fig. 3. Finally, motion-activated systems refer to tuned mass dampers. These devices divert input energy from the main structure to a secondary mass—typically hanging from the highest level—whose inertial forces dissipate energy.

3.2 Base isolation

Base isolation was the first developed SPT, and actually, the first application was for a wooden house in 1885 (Naeim and

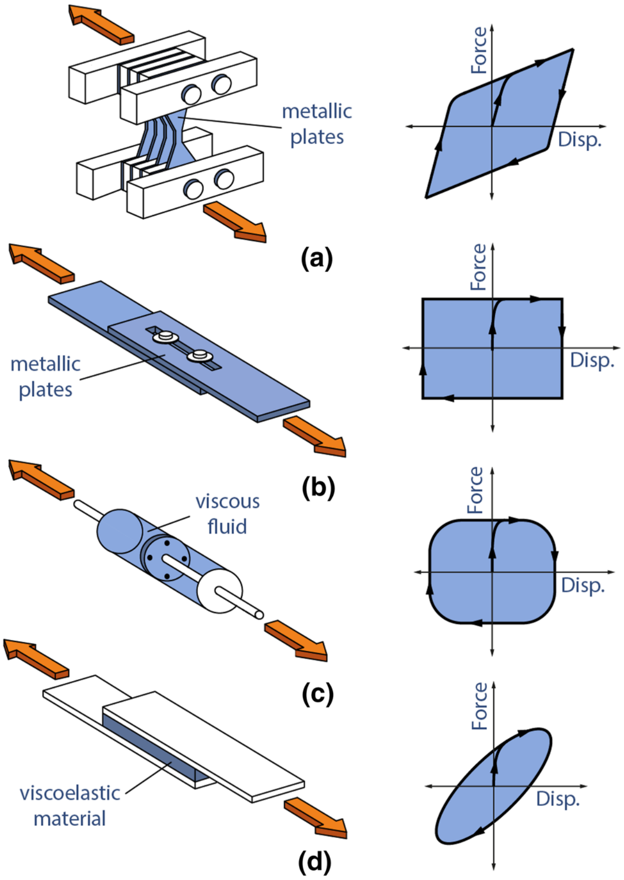

Fig. 3 Schematic configuration and idealized force–displacement hysteretic behavior of **a** metallic damper, **b** friction damper, **c** fluid viscous damper and **d** viscoelastic damper

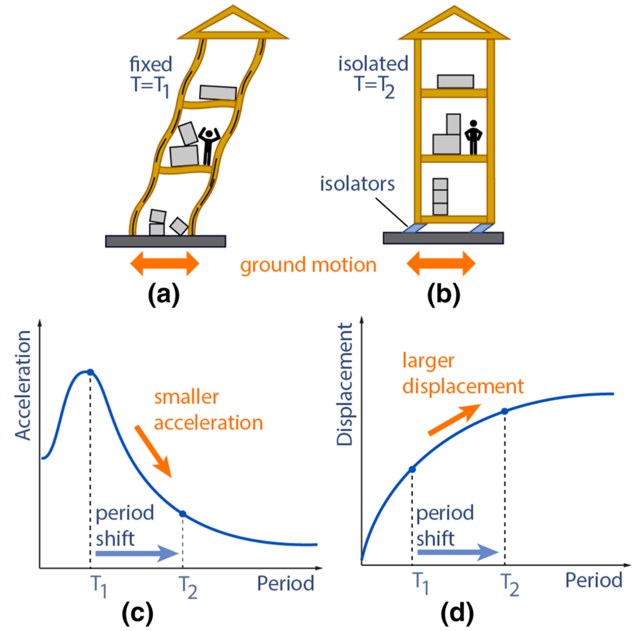

Fig. 4 Schematic seismic response for a structure with **a** fixed base and **b** isolated base. Effect of the period shift in structural response for **c** acceleration and **d** displacement

Kelly 1999). However, it was not until the second half of the twentieth century when new materials and technologies allowed for more efficient isolators. In terms of Eq. 2, base isolation aims at reducing input energy to prevent dissipation of exceedance through damage. In the rain flow analogy, base isolation is equivalent to close the stopcock in the plumbing to drastically reduce the water entering the system. Such reduction is given by the utilization of a flexible interface (isolators) beneath the supports of a structure (superstructure) such that structural response turns uncoupled from ground motions (Fig. 4a, b). This SPT results in much more flexible structures with significantly increased periods (typically 1.5–2 s) being subjected to much smaller lateral demands, especially observed through much smaller accelerations of the superstructure (Fig. 4c) letting acceleration-sensitive components undamaged. Opposite to supplemental damping, the period shift increases the total displacements during an earthquake (Fig. 4d), however, most of this displacement concentrates in the isolation system (Fig. 4b), while the superstructure typically shows much smaller relative deformations (up to 95% compared to the unprotected case) and so less damage in structural and non-structural components. As the superstructure supports over the isolators, these must withstand overturning and be able to transmit gravity and vertical seismic loads to the foundation. This is critical during an earthquake, since the isolators are subjected to large lateral deformations that substantially increase buckling risk due to vertical loading. The isolation system must also provide either a minimum initial stiffness or activation force, in order to prevent movement arising from wind loading or small ground motions. Furthermore, flexible joints must be accommodated to installation services at the interface between the superstructure and the ground.

The main types of isolation systems include: (i) rubber or elastomeric isolators, i.e. cylinder-shaped members with circular layers of rubber, typically interleaved with steel layers to increase vertical stiffness (Fig. 5a). Similarly, elastomeric isolators in long rectangular strips with multi-layer fiber-reinforcement have also been studied; (ii) lead-rubber isolators, distinguishing from the former in that a lead core increases energy dissipation of the system (Fig. 5b). They may include natural (less variable properties) or high-damping rubber (higher damping ratios); (iii) flat slider bearings, consisting of steel plane surfaces over the ones a slider component moves overcoming the friction between both surfaces; (iv) friction pendulum sliders (FPS), similar to the previous except that the sliding surface is curved, typically spherical (Fig. 5c); and (v) roller bearings consisting of rails or spheres over the ones the structure slides (Fig. 5d). Vertical springs have also been used to isolate mechanical

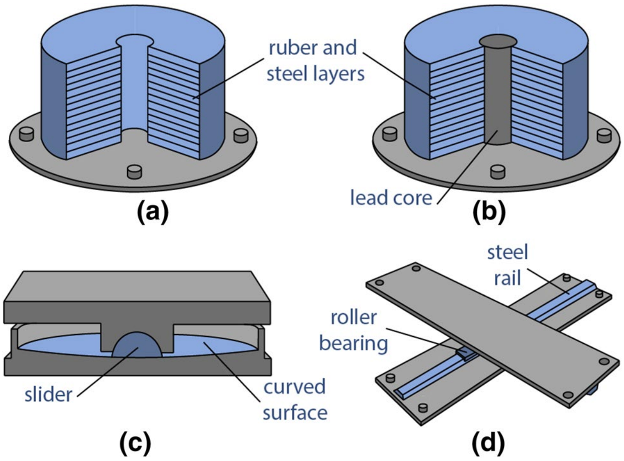

Fig. 5 Schematic section of **a** rubber isolator, **b** lead-rubber isolator, **c** FPS and **d** roller bearing

equipment mainly. Most of the previous isolation systems have a restoring or self-centering force, i.e. a mechanism to restore the undeformed position of the isolator after motion. The restoring force may be provided by the elastic stiffness of a rubber isolator or by the curvature of an FPS. On the other hand, flat sliders or roller bearings do not have restoring force and need to be combined with other isolators. For FPSs, the period depends on the slider curvature rather than the structural mass, what turns very advantageous for isolating light timber structures.

3.3 Rocking systems

A rocking system is a SPT in which one or more structural parts can, up to some extent, rotate relative to each other like a rigid body while the structure is laterally loaded, being such monolithic rotation referred to as rocking. Often, but not always, these SPTs are accompanied by post-tensioning (PT) technologies, i.e. post-stressed steel tendons, that, along with gravity and timber elastic deformation, provide restoring force to un-rocked configurations. In terms of Eq. 2, rocking systems typically reduce the potential energy in terms of elastic deformation, but total potential energy tends to increase due to uplift and deformations of PT systems. In general, rocking systems modify energetic balance in that capacity of kinetic and potential energies of the building increase as well as the inherent damping energy flow. Damping here is mostly provided by the 'touch' of assemblies while rotating. However, supplemental damping is normally used, such that the monolithic displacement is used as input for other displacement-based dampers to significantly increase the overall damping. The typical configuration of rocking walls and frames containing PT tendons and supplemental dampers is shown in Fig. 6a. Dampers in walls are

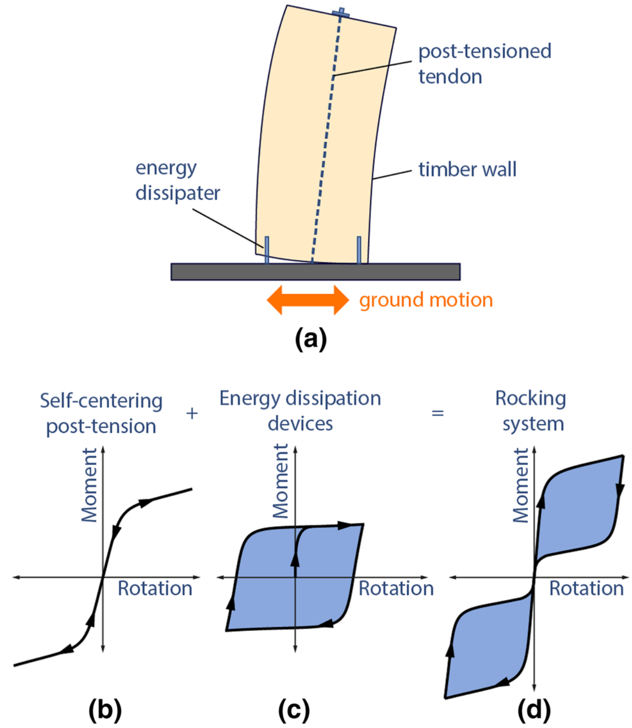

Fig. 6 **a** Schematic configuration of a typical PT rocking wall. Hysteretic behavior of **b** an elastic self-centering system, **c** an energy dissipation system and **d** the resulting rocking system

typically located either close to hold-downs to benefit from uplift displacement or in-between twin walls to use relative wall displacement, while dampers in moment frames are placed in the beam to column connections (BTCs) given the large compression forces produced by PT tendons and the need for large stiffness to produce monolithic rotation. This kind of SPT has mainly focused so far on mass timber walls made from Laminated Veneer Lumber (LVL) and Cross-Laminated Timber (CLT) as well as stiff glulam or LVL moment resisting frames.

The force–displacement relationship of a typical PT rocking wall or BTC system is presented in Fig. 6. The timber assembly and tendons are expected to behave solely elastic, thus providing restoration (self-aligning) forces, as presented in Fig. 6b. The second (inelastic) branch is given by the yielded tensioned system, plus the lateral resistance of the timber assembly with the entailment of the damping system which comprises structural stiffness against uplift. The dissipative behavior is provided by dampers—typically metallic or friction dampers—as shown is Fig. 6c. In the composed system, Fig. 6d, large energy dissipation is released upon activation force or moment, casting a two symmetric flag-shaped hysteresis curve. Note that an additional advantage of rocking systems for wall components is that tensile forces are highly alleviated during uplift at wall ends.

4 Review of seismic protection with supplemental damping

Most supplemental damping research in timber structures involved light-framing shear-walls. Filiatraut (1990) began with the first studies by bracing the 4 corners of a light-framed shear wall with friction dampers (Fig. 7a). Numerical models were validated with existing shaking tests of conventional panels, and then used in time history analyses (THA). Dampers dissipated 60% of the input energy reducing peak forces and displacements. Dinehart and Shenton (1998) and Dinehart et al. (1999) tested four different configurations of viscoelastic dampers in light-framed walls (Fig. 7b–e): (1) at upper corners, (2) sheeting to stud, (3) in diagonal bracing and (4) in cable and pulleys arrangement. The walls with dampers showed larger energy dissipation capacities, up to 55% larger for the diagonal bracing. Further tests of viscoelastic materials were presented by Dinehart and Lewicki (2001), proofing the feasibility to connect them directly to the wood, as failure of dampers was always observed within the viscoelastic material (Fig. 7f). A full-scale wall with an extremely thin layer of viscoelastic material (0.0127 mm) between the sheeting and the studs was cyclically tested, showing a 26% increase in the energy dissipation capacity. Higgins (2001) tested a diagonal damper composed of a steel bar that dissipated energy by tension yielding, and whose lower end is allowed to slide when subjected to compression, thus avoiding buckling (Fig. 7g). Cyclic tests showed that the special diagonals prevented stiffness degradation while THA with these devices showed a 50% reduction in peak displacements.

Fluid viscous dampers in diagonal configuration (Fig. 7h) were analyzed by Symans et al. (2002a, c) through FE models of light-framed walls and a 2-story building, whose increase in the dissipated energy prevented the collapse observed in the model without dampers. Du (2003) and Symans et al. (2004) numerically assessed the effectiveness of fluid viscous dampers in diagonals of light-framed shear walls. The models simulated both single walls and a 2-story buildings showing large reductions in drifts (57–91%) preventing model collapse when fluid viscous dampers were added. Such enhanced performance was subsequently validated by Dutil and Symans (2004) via shaking table tests on light-frame walls but placing the dampers horizontally in chevron braces (Fig. 7i), finding that drift was reduced by 51% while energy dissipated by the wood frame drooped 71%. The use of splice plates, pre-stressed with bolts to increase friction damping in beams was proposed by Awaludin et al. (2007). Quasi-static cyclic and shaking table tests showed an increase in damping (42%) and reduction in rotation demands (27%). Pre-stress force was monitored during a year showing a large loss (77%).

The 1994 Northridge earthquake motivated large and important projects for the further development of supplemental damping in light-frame construction. The NEES-Wood Project (Filiatrault et al. 2007, 2010) comprehended shaking table tests of a full-scale two-story structure with wood garages in the first story, a layout commonly observed in damaged buildings of the US. The experimental program included a prototype equipped with fluid viscous dampers as presented by Shinde et al. (2007). Viscous dampers

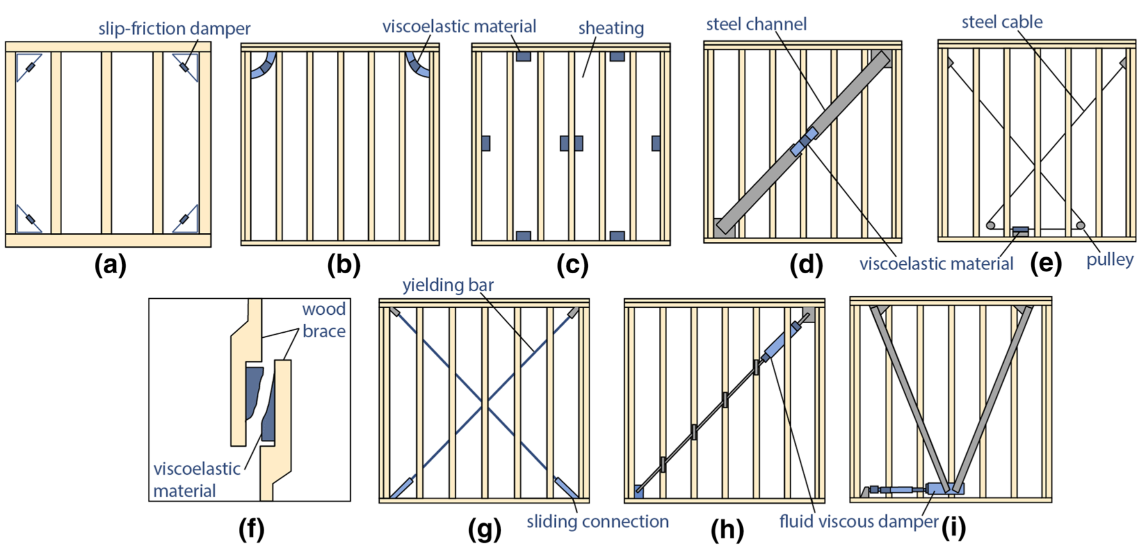

Fig. 7 Different tested configurations of supplemental damping in light-framed shear walls, based on the research from: **a** Filiatrault (1990), **b–e** Dinehart et al. (1999), **f** Dinehart and Lewicki (2001), **g** Higgins (2001), **h** Symans et al. (2002a) and **i** Dutil and Symans (2004)

connected to metallic chevron braces within wood frames. Although numerical simulations predicted the potential benefits of such design, the limited displacements in the damper during the shaking table test reduced its effectiveness. In order to amplify these displacements, Shinde et al. (2008) employed a well-known toggle brace configuration (Fig. 8a), which was implemented in numerical models of the NEESWood prototype building to evaluate its seismic performance. Numerical and experimental results were reported by Shinde and Symans (2010) along with design recommendations. Retrofitted walls reduced drift (78%) and energy dissipation (73%) demands. Improvements were attributed to larger stiffness and damping. However, the flexibility of wood connections reduced the effectiveness of dampers about 30 to 40%. An additional project, entitled NEES-Soft Project (van de Lindt et al. 2013), further covered the modeling, design and retrofitting of this kind of structures. The program included tests of a wall with and without toggle-braced fluid dampers (Shao et al. 2014) using real-time hybrid simulation, i.e., pseudo-dynamic tests of certain components and simultaneous numerical simulation of the remaining structure. The drift of the damped specimen decreased 32%. Additional tests with toggle brace systems included a slow hybrid simulation of a three-story building (Tian et al. 2016) where energy dissipated by dampers was twice that dissipated by the wood framing. Additionally, shaking table test of a four-story building with toggle brace dampers (Bahmani et al. 2014; Tian et al. 2014; van de Lindt et al. 2016) showed that it is more effective to distribute the seismic retrofit (and demand) in several floors of the building than to concentrate it on the first floor. Based on these results, Tian (2014) presented an analysis to assess

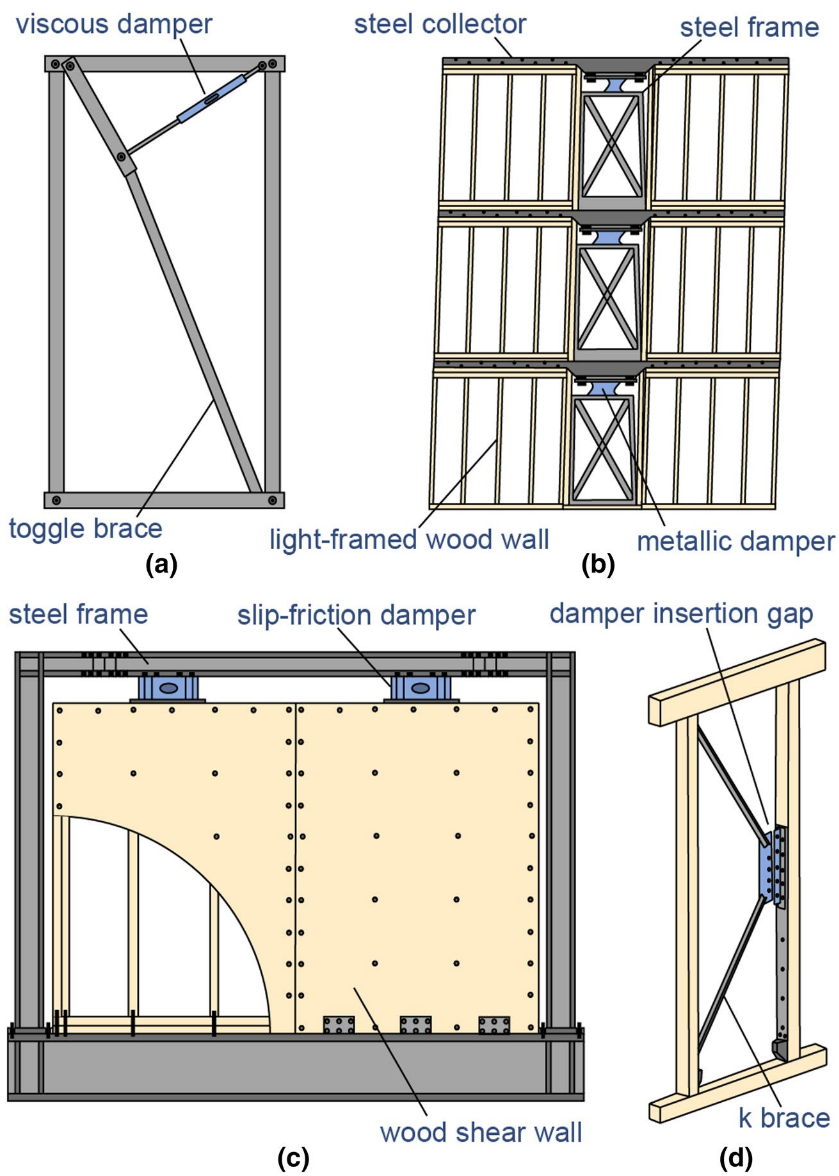

Fig. 8 SPT based on supplemental damping. Adapted from **a** Shinde et al. (2008), **b** López-Almansa et al. (2015), **c** Li et al. (2017), **d** Kasai et al. (2005)

the effectiveness of energy-dissipation retrofit procedures through a performance-based-design (PBD) approach. She also extended an existing displacement-based design methodology to consider the mitigation of torsional response with an optimized planar distribution of dampers. A similar method for optimal height damper distribution was presented by Pu et al. (2016) while Pu et al. (2018) proposed a methodology to determine the optimum capacity of hysteretic dampers for multi-story timber structures.

Further research on light-framing was carried out in New Zealand and China. López-Almansa et al. (2015) proposed the protection of multistory light-framed buildings by adding steel braced frames connected to the slabs through steel collectors, see Fig. 8b. Supplemental damping was accomplished by yielding metallic plates that were located in between the collectors and the braces. The system was modeled in a six-story building in New Zealand, designed to accomplish performance-based limit states, obtaining reasonable plate thickness (8 to 12 mm). A similar approach was performed by Li et al. (2017) who presented steel frames with infill wood shear walls, connected through friction dampers to dissipate energy (Fig. 8c). The study included wall tests, numerical models and a design example for a four-story building in a seismic prone area of China, which due to the dampers reported reductions up to 37% in drift and 25% in floor accelerations.

In addition to light-framing, extensive studies in Japan have focused on the protection of post-and-beam buildings, due to its importance in traditional Japanese wood housing, as it showed a very deficient performance during the 1995 Kobe earthquake (Prion and Filiatrault 1996). Kawai et al. (2006) proposed to use dampers made of viscoelastic polymers. At the Tokyo Institute of Technology, Kasai et al. (2005) developed the so-called k-brace system (Fig. 8d) which dissipates energy through the vertical displacement of friction or viscoelastic dampers located between the brace and the frame. Experimental studies included cyclic dynamic tests (Matsuda et al. 2008a) and shaking table tests for single-story frames (Sakata et al. 2007) as well as two-story frames (Sakata et al. 2008). The latter tests were extended by Matsuda et al. (2008b) who included specimens with exterior partitions finding that the undamped wall showed a large fall in the frequency properties after the shaking tests, suggesting larger damage. In order to control torsional rotations, Yamazaki et al. (2010) performed shaking table tests of single-story frames with several in-plane arrangements of viscoelastic dampers, located either in k-braces or at the upper corner of the frames, being the first one the most effective. Matsuda et al. (2010, 2012) presented an analytical model composed of several spring elements to represent the nonlinear response of k-braces, which were more effective than plywood walls in reducing maximum displacement in THA. The insertion of viscoelastic or metallic dampers in between plywood panels was presented by Sakata et al. (2017) finding the first ones to be more effective at low rotations. The study that also included k-braced frames proposed a design method for wooden houses. Xie et al. (2018) studied the use of vertical metallic bars inserted in complex beam-to-column connections of traditional Asian buildings, in order to increase energy dissipation and self-centering capacities.

Some recent research on supplemental damping has focused on CLT buildings. Poh'Sie et al. (2016) carried out analytical models of several arrangements of tuned mass dampers. The optimal designs were implemented into a finite element model of a full-scale seven-story CLT building previously tested without dampers as part of the SOFIE project in Japan (Ceccotti et al. 2013). The high floor accelerations recorded during the shaking table tests in the upper stories were reduced up to 38% in the simulations. Polocoşer et al. (2018) tested a beam-to-column connection composed of steel plates and pre-stressed bolts whose sloths have a larger diameter to allow for slippage and the consequent frictional energy dissipation. Shaking table prototypes showed good performance although a prefabricated joint was recommended to guarantee the adequate pretension of the bolts. Yousef-beik et al. (2018) presented the use of a resilient slip-friction joint in timber bracing systems. These devices consist of steel sliding plates with grooved surfaces that increase energy dissipation and provide a restoring force due to pre-stressed bolts and Belleville washers. Analyses and tests showed adequate performance, as long as the buckling of the brace is prevented.

5 Review of seismic protection with isolation

The first research on isolated wooden structures concentrated on elastomeric isolators. Delfosse (1982) presented the design of an elastomeric isolation system for a single-story light-framed house, designed to keep elastic response of the superstructure. Based on this pioneering research, he remarked implications of isolating light-framed wood structures, given that the low mass reduces the period shift, and therefore isolation effectiveness. In addition, the period shift requires a very flexible isolator, leading to very slender devices whose buckling is hard to prevent. Reed and Kircher (1986) explored the retrofit of an existing five-story building using base isolation. The 100-year-old light-framed structure was analyzed both with elastomeric bearings and flat slider bearings, showing 74–98% reduction in the base shear during THA. Sakamoto et al. (1990) performed free vibration tests of a two-story house isolated with several kinds of rubber bearings, including high-damping, lead-core, and steel layered (Fig. 9a). For the latter bearings, numerical simulation and shaking tests were also performed, showing a 70% reduction in the ground accelerations transmitted to the first floor.

More recent research on isolation included other types of isolators in an attempt to avoid the problems mentioned above. Pall and Pall (1991) implemented base isolation on an actual two-story light-framed house in Montreal, Canada, which was supported over flat sliding bearings along the perimeter of the basement wall, reducing about 42% the accelerations on the structures. Zayas and Low (1997) isolated a four-story light-frame building in San Francisco, California. After severe damages in the Loma Prieta earthquake, the structure had its first story replaced by steel frames supported on an FPS isolator benefitting in that such SPT is non-dependent upon the mass. Non-linear THA showed a 95% reduction in story drifts given by the isolation system. Iiba et al. (2000) conducted 3D shaking table tests of 7 types of base isolators placed under a loaded platform in order to analyze its implementation in wooden houses in Japan. The isolators included rubber bearing, sliders, and rolling systems, as well as a combination of them. Bi-directional input and vertical acceleration were evaluated finding no significant effects, likewise for mass eccentricity, which produced small torsional rotations (less than 0.75 deg). The same analysis was presented by Myslimaj et al. (2002) but focusing on double spherical FPS isolators (Fig. 9b) reporting response accelerations 50–80% smaller than input accelerations. This system had previously been implemented by Iiba et al. (2001) in a full-scale two-story house instrumented to perform cyclic tests and monitor dynamic response. Recognizing that the low mass of wood structures could lead to the slender design of rubber bearings, Iiba et al. (2004) presented a buckling protection plate (Fig. 9c) and a displacement restraint device (Fig. 9d). These devices improved

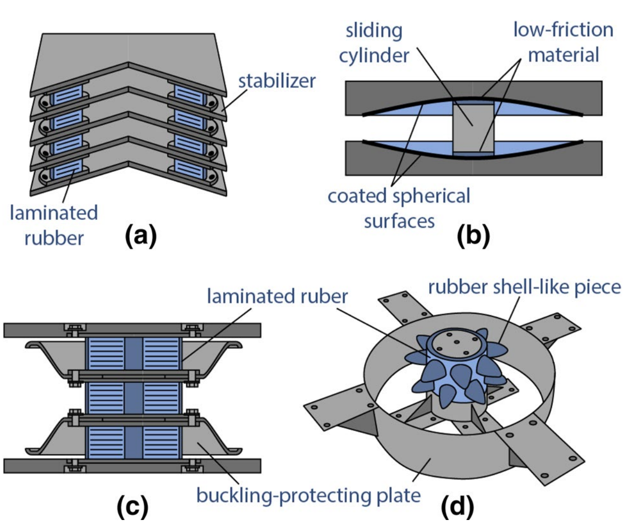

Fig. 9 SPT based on isolation. Adapted from **a** Sakamoto et al. (1990), **b** Myslimaj et al. (2002), **c, d** Iiba et al. (2004)

the performance of the rubber bearings in numerical simulations and shaking table tests where response accelerations were 75% smaller than table accelerations. Liu et al. (2009) presented the design of two light-framed wooden structures (two and six stories) isolated with sliding bearings. The distribution of isolators and the effect of the isolators' radii of curvature were assessed through PBD criteria, showing that small radii (460 mm) lead to unsafe drift demands, while larger radii (> 1880 mm) perform much better- although it implies much larger costs. As part of the aforementioned NEESWood project, Shinde and Symans (2010) presented shaking table tests of a half-scaled two-story light-framed house supported on FPSs. Two methodologies for PBD on a displacement-based approach were proposed: one using a single-degree-of-freedom system, and another through a simplified modal analysis. The project concludes that for reasonable design drift ratios, for example, 0.005, it is feasible to analyze the structure as a rigid body aiming for an operational performance level. This research was also presented by Van de Lindt et al. (2011) highlighting construction issues of providing in-plane stiffness to the floor diaphragm with steel beams. Models -validated with tests- showed up to 70% reduction in inter-story drifts with the isolation system. Van de Lindt and Jiang (2014) performed a regression analysis using models of 6 multistory light-frame buildings isolated with FPSs, to propose an equation that relates the required radius of curvature to relevant parameters of demand and expected performance.

Many researchers stressed the need to reduce the cost of isolators, and as so several modern researches have investigated the use of alternative materials, mostly in sliding or FPS isolators. Jampole et al. (2014) tested several combinations of materials and shapes for sliding isolation systems. To reduce costs, common inexpensive materials like steel, polyethylene or nylon were evaluated as sliding surfaces, aiming for large friction coefficients (0.15 to 0.25) that reduce the displacements of the slider and consequently the required size of the sliding plate. The tests included flat and concave plates, the latter offering larger slip resistance to reduce displacements and self-centering force to limit residual deformations. Although reducing displacements increased base shears to values as large as 0.275 g, this isolation system was proposed along with the so-called "UniBody" light-frame construction (Swensen et al. 2014) which fully engage frame and sheeting, offering larger strength than conventional light-frame system. Flat and concave plates made of the selected materials (i.e. galvanized steel for sliding plates and high-density polyethylene for sliders) were successfully implemented in shaking table tests of a two-story UniBody house (Jampole et al. 2016) (Fig. 10a). Fourteen MCE-level ground motions were applied. Reported base shears were larger than expected (0.38 g) but keeping small drift ratios (0.09%) and elastic

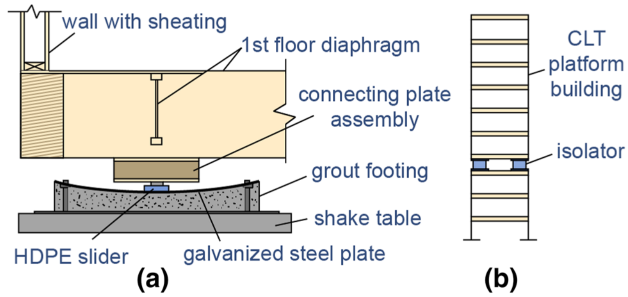

Fig. 10 SPT based on isolation. Adapted from **a** Jampole et al. (2016) and **b** Bolvardi et al. (2018)

response. Computational models to reproduce the tests were presented by Jampole et al. (2017).

Last research on isolation has investigated the efficiency of elastomeric isolators in mass timber buildings, as mass and rigidity of such buildings are closer to steel and concrete buildings. Bolvardi et al. (2018) proposed the use of inter-story isolation for CLT platform buildings (Fig. 10b) using displacement-based design. The method was implemented in a 12-story building assumed in Los Angeles, California. The building performance was assessed through direct-displacement-design meeting the established limit displacements and reducing the inter-story drift ratios obtained in the non-isolated structure. Furthermore, locating the isolation system in higher levels reduced isolators displacement but increased inter-story drift.

6 Review of seismic protection with rocking systems

The self-centering principle behind the rocking systems was firstly introduced in the concrete by the PREcast Structural Seismic Systems (PRESSS) program (Priestley 1991). This system consisted of PT joints with unbonded tendons to provide self-centering force and some source of energy dissipation. Because of these two effects, it was termed as hybrid system. Within the same program, Kurama et al. (1999) focused on the behavior of rocking walls proposing limit states for PBD. Kurama (2000) also remarked the disadvantage of the larger drifts on rocking walls, which is reduced with the inclusion of supplemental damping.

The PRESS system was extended to LVL timber structures at the University of Canterbury (Palermo et al. 2005, 2006b, c). The program included quasi-static tests of several hybrid connections for beam-column connections, BTC, (Fig. 11a), showing non-linear elastic behavior with an equivalent "yielding point" corresponding to the uplift onset. While initial stiffness was independent of the PT force, yielding point and the post-yielding stiffness increased for higher PT forces. Without dampers, energy dissipation was very small; however, with the addition of internal metallic dampers, the expected flag shape with relevant hysteresis dissipation was recorded. Assemblies were tested up to 4.5% drift ratio without visible damage, and equivalent yielding point was observed about 0.5%. As part of the same program, Iqbal et al. (2008) performed bi-directional tests on column-to-foundation PT connections that dissipated energy through yielding of mild bars externally attached (Fig. 11b). Pseudo-dynamic tests reported drifts about 3% with no significant damage. The same concept was applied by Iqbal et al. (2010) who tested BTC joints, again with dissipaters located externally to facilitate replacing. Dynamic tests of PT frame buildings without dissipaters were presented by Pino et al. (2010) and Pino (2011). Tests consisted of three- and five-story buildings (1/4-scale), which allowed computing equivalent viscous damping ranging from 2% for low drifts to 8% for the larger ones. Quasi-static tests, numerical models and design examples were also presented. An alternative design was modeled and tested by Smith et al. (2012b, 2014b) who used glued-laminated BTC connected with steel angles for energy dissipation and a steel tube glued into the beam as a shear key. Equivalent viscous damping ratios obtained from models and tests ranged between 6 and 17%. This connection detail was implemented in a 3-story building modeled by Smith et al. (2012a) and tested on a

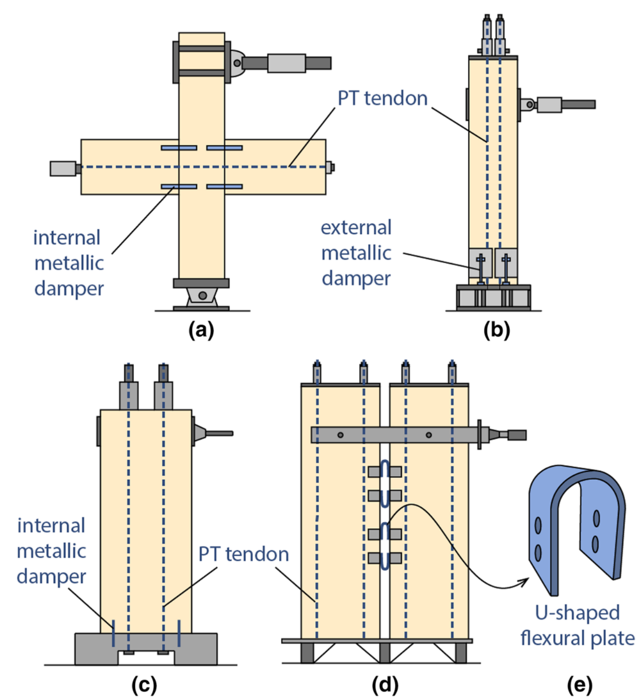

Fig. 11 Test assembly for rocking systems adapted from **a** Palermo et al. (2005) for beams, **b** Iqbal et al. (2008) for columns, **c** Palermo et al. (2006a) for single walls and **d** Iqbal (2011) for coupled walls using, **e** U-shaped flexural plates (Sarti et al. 2016)

2/3-scaled building by Ponzo et al. (2012) and Smith et al. (2014a). In these tests, the addition of steel angles reduced drift ratios 32% without increasing floor accelerations. Based on these tests, further non-linear models in commercial- and research-oriented software were presented by Simonetti et al. (2014) finding differences of less than 25% compared with tests results. In addition, Smith et al. (2016) tested a 2-story full-scale building to assess the lateral response of PT frames designed for gravity loads. Shu et al. (2018) presented an analytical comparison of a conventional braced frame structure with a post-tensioned self-centering timber frame structure, which showed better seismic performance although some challenges are highlighted, such as damage in connections and non-structural components.

A known issue of PT timber structures resides in long-term effects, and particularly in the loss of PT force related to timber rate effects as well as its dimensional instability, which was investigated by Davies and Fragiacomo (2011) for LVL beams during 1 year under controlled and uncontrolled conditions. PT forces decreased 1.4% for beams loaded parallel to the grain, and 7% for frames whose columns are loaded perpendicular to the grain. An analytical model to predict untightening in time depending on the geometrical and structural parameters was derived by Fragiacomo and Davies (2011). The model predicted after 50 years a 6% PT loss for a member loaded parallel to the grain, but the loss increased up to 38% and 71% if 11% of 100% of the assembly length was loaded perpendicular to the grain, respectively. This model showed good agreement with tests measurement by Granello et al. (2017) who monitored PT beams to assess the long-term behavior after 4 years, finding PT force losses between 3.4 and 4.5% for loaded and unloaded beams, respectively. Granello et al. (2018) proposed a design method for estimating the amount of post-tensioning loss expected over the service life of the most common configurations of frames.

Post-beam rocking assemblies have also been studied in Germany and Switzerland. Kasal et al. (2014, 2015) proposed a frictional BTC, which was tested under cyclic load and later mounted on a 1:3 3-story scaled structure on a shake table. Although the BTC showed very large dissipation capabilities, and the timber elastic energy was enough for shape recovering without the need of a PT system, drift and friction onset were difficult to control and ultimately the structure failed in a brittle. This research proofed the difficulty of finding a balanced compromise between rigidity for drift prevention, strength and ductility for timber moment resisting frames. Wanninger and Frangi (2014) used glulam spruce members strengthened with hardwood at the BTC where compression perpendicular to grain was expected. The beams were bearing on notches made on the column, and therefore the bottom of the beams' ends was also strengthened with hardwood. Even with this strengthening, the decisive design criterion was the strength perpendicular to the grain leading to a ductile failure. This configuration was implemented in the ETH House of Natural resources, which was instrumented for modal data acquisition by Leyder et al. (2015b) and health-monitoring by Leyder et al. (2015a) finding PT force losses between 6 and 12% in the first 3 months for controlled and uncontrolled conditions, respectively. Similarly, the long-term behavior of this system was analyzed in lab specimens by Wanninger et al. (2015) finding losses of PT force in the range of 5 to 10% after 1 year, suggesting that it can increase up to 30% after 50 years, depending on environmental and geometrical conditions. Further results after another year of measurements were presented by Wanninger (2015), who also performed pushover tests of 1-story frames and presented models to reproduce the behavior and the PT losses. He verified that PT force increased and decreased along with the relative humidity. Keeping constant humidity losses ranged from 15 to 20%, while in the uncontrolled environment losses ranged from 25 to 30%. For design, a 30% loss was recommended.

The first tests on timber rocking walls were presented by Palermo et al. (2006a) who used internal and external dissipation bars (Fig. 11c) obtaining behavior similar to that previously observed in BTC with no visible damage for interstory drifts as large as 4.4%. Smith et al. (2007) extended this study to coupled walls, equipped with external metallic dampers and externally nailed plywood sheets in order to dissipate energy; 2.5% drift ratios were achieved without noticeable damage. For the scheme with plywood sheets, additional tests and an analytical model were proposed by Iqbal et al. (2012) finding good energy dissipation capacities but less damping than other connections. Iqbal et al. (2007) proposed the use of external U-shaped Flexural Plates (UFPs, Fig. 11e) between adjacent walls to dissipate energy, being these devices already used in concrete, becoming later one of the most widely used in timber rocking systems. Iqbal (2011) performed quasi-static and pseudo-dynamic tests of 11 LVL walls, including specimens with and without external bar dissipaters and some coupled with UFPs (Fig. 11d). The later dissipation system was slightly more efficient in terms of larger equivalent viscous damping (7% for bars and 9% for UFPs) and smaller lateral displacement (33 mm for bars and 24 mm for UFPs).

Newcombe et al. (2010b) presented the design, fabrication and cost evaluations of the first full-scale building prototype, a 2-story LVL building made of PT frames in one direction and rocking walls equipped with UFP devices in the other. The wall systems were more cost-effective than frames for resisting lateral loads, mainly due to the high cost of external rods. The prototype was assembled in 17 h and 80% of the costs were related to materials and pre-fabrication, verifying on-site efficiency. Quasi-static tests of this building were presented by Newcombe et al. (2010c), assessing the yield of dampers and the influence of diaphragms. No significant damage and full re-centering were observed up to 2% drift, however one column fractured at 3% drift due to a localized connection detailing. With the addition of concrete slabs, strength increased 15% in the frame direction and 25% in the wall direction. This work was extended by Newcombe (2011) with analytical and numerical models to reproduce the building tests and applying displacement based design procedures to walls, frames, and diaphragms.

Design aspects were first discussed by Pampanin et al. (2006) with a PBD approach. Newcombe et al. (2008) presented a simplified method based on existing procedures for precast concrete but considering the orthotropic behavior of timber materials. Newcombe et al. (2010a) applied displacement-based design to a timber frame. Sarti et al. (2012) also presented a simplified approach for displacement-based design of LVL rocking walls developed through a parametric study. The simplified method was proved on a hypothetical 21 m tall building showing good consistency with a more sophisticated model. Newcombe et al. (2012) made a comparison of the governing lateral loads for PT timber structures in New Zealand, finding earthquake loading as the critical design parameter in terms of displacement, while wind may produce larger force demands in certain regions of the country. Force- and displacement-based design procedures were presented by Sarti (2015) including implementation of FEMA P695 methodology using proposed numerical and analytical models validated with connection and wall subassemblies. A response modification factor $R=7$ was recommended for dissipative post-tensioned walls.

Smith (2008) and Smith et al. (2009) presented a comparative analysis between a concrete building with hypothetical alternative buildings made of steel and PT timber. The comparison was made in terms of member size, construction time and cost. Concrete and timber structures had similar member size and construction time. While cost varied less than 1% between steel and concrete buildings, the timber solution was approximately 6% more expensive, and up to 11% more expensive for the alternative with additional timber architectural features. Most of the difference is due to the cost of structural elements and flooring systems. Devereux et al. (2011) and Holden et al. (2016) documented the first implementation of timber rocking systems in an actual building in Nelson, NZ. This 3-story LVL structure used PT rods and UFPs, and was assessed through PBD. Palermo et al. (2012) showed the design and construction of an auditorium in Carterton, NZ, with 11 rocking walls of 6.7 m tall. These LVL walls used bars for post-tension and internal bars grouted into the wall as dampers. Analytical models were presented for pushover and non-linear THA of this structure. Another implementation was presented by Brown et al. (2012) in the Trimble Building in Christchurch, NZ. Dunbar et al. (2013, 2014) proposed testing and design detailing of two half-scale CLT core-walls post-tensioned with a single 15.2 mm tendon. Details are presented for low and high seismic areas, the latter including steel columns at the corners and UFPs between walls.

Regardless of the material and the base connection (post-tensioned or monolithic), walls, in general, showed vertical displacement incompatibilities with floor diaphragms, and several connection details have been proposed to handle this problem. Particularly for rocking timber walls, Devereux et al. (2011) connected beams around the walls using large diameter pins to transfer lateral loads while allowing rotations (Fig. 12a). Brown et al. (2012) approached similarly but using slotted holes to allow vertical displacement. These and other five connection details were tested by Moroder et al. (2014), concluding that floor damage can be prevented by the flexibility of well-designed connections and beams. However, special attention must be paid when stiff floors or beams are used. Sarti et al. (2016) also used dowels to allow rotation of collector beams, but these were connected to boundary columns rather than the wall (Fig. 12b). In the gap between the columns and the wall, sliding LVL bearing blocks were located to transfer horizontal shear forces and UFPs to couple and dissipate energy. As similar displacement incompatibilities may be observed horizontally in frame systems, Moroder et al. (2013) performed an experimental campaign on mechanisms to prevent damage in diaphragms through concentrated or distributed floor gaps. Pei et al. (2018) detailed the connection between a rocking wall and the diaphragm using a dowel within a vertically slotted hole to transfer lateral forces while allowing for the relative vertical movement. This connection was implemented in a shaking table test of a 2-story structure whose rocking wall was only damaged at the intended replaceable locations while the gravity frame and diaphragm (including their connections) remained undamaged.

Ganey (2015) stressed on the design of PT walls equipped to rock on intermediate stories and tested five specimens with different details, showing good ductility and energy dissipation due to UFP devices. Shear keys were included at the base of the walls to control rocking. Tests showed ductile behavior with significant PT force lost due to damage, which before 5% drift did not affect the re-centering. UFP devices provided 50% of the energy dissipation. The analysis included the PBD of two buildings located in Seattle, USA (8 and 14 stories) with rocking stories at the bottom and intermediate stories. Similar to inter-story isolation mentioned before, the lower the rocking story was located, the better performance was observed. Intermediate joints were proposed for rocking systems by Wiebe et al. (2013) in order to limit high mode effects, i.e. an increase in demands due to secondary vibration shapes.

Ma (2016) presented an alternative system with CLT walls pinned at the base, coupled with yielding dampers and

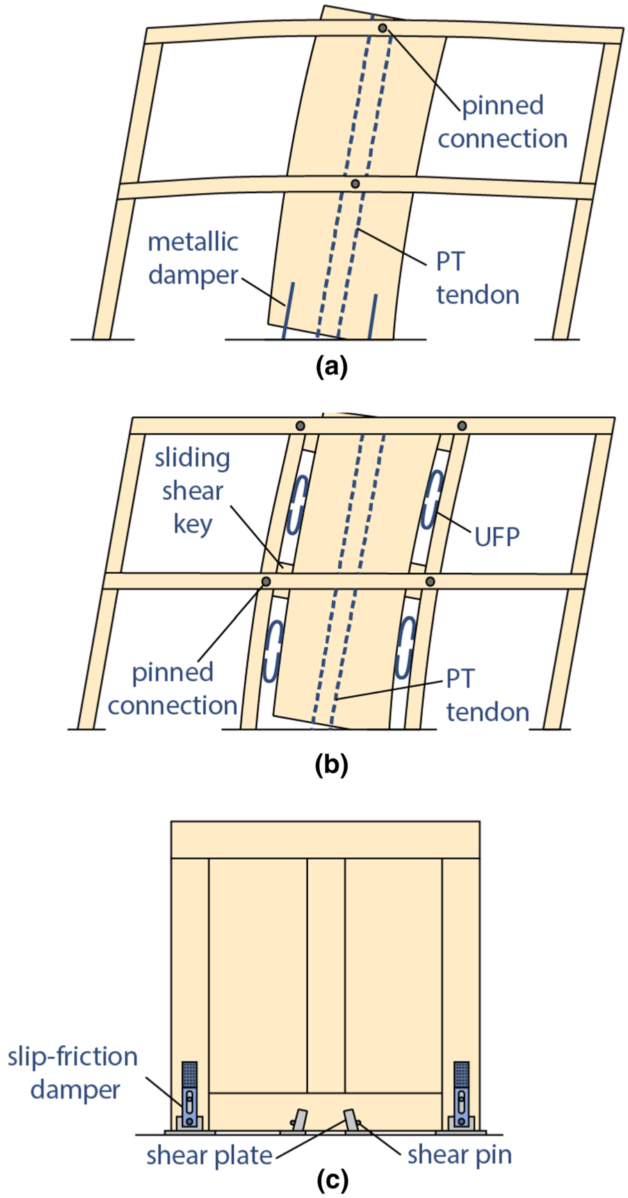

Fig. 12 **a, b** Frame to rocking wall connections adapted from Sarti et al. (2016). **c** Rocking wall with slip–friction damper adapted from Loo et al. (2014)

without PT. Numerical models were presented and applied to a 6-story building example in Vancouver, Canada. The system performed well in terms of maximum drifts and base shears in THA, but some residual drifts were larger than the desired value because restoring forces were not large enough. Akbas et al. (2017) conducted six cyclic tests on CLT rocking walls post-tensioned with bars (two of them connected with UFPs). Six progressive limit states were proposed: (1) base decompression; (2) linear-elastic limit; (3) CLT yielding; (4) CLT splitting; (5) CLT crushing; and (6) bars yielding. Simplified design equations and finite elements were applied, finding good agreement with the tests.

Zimmerman and Mcdonnell (2017) showed some considerations behind the PBD of a 12-story CLT building to be located in Portland, USA (high seismic area), highlighting the relevance of considering effective shear modulus, high mode effects and deformation compatibility of gravity connections. For regions of moderate seismicity, Kovacs and Wiebe (2017) proposed a force-based design method of CLT walls with PT bars at the boundaries to control rocking, and without supplemental damping. The method, which included high-mode considerations, was used to design wall prototypes of 3-, 6- and 9-stories in Montreal, Canada. All models had less than 5% probability of exceeding the shear or bending moment capacities at the maximum considerable earthquake level. However, the 3-story building exceeded the allowable probability of collapse, i.e., 10%.

Morrell et al. (2018) developed alternative inter-panel connectors designed to act as energy dissipation fuses. These devices consisting of steel stripes fastened to adjacent panels are intended to provide large initial stiffness and deformation capacity. The system was implemented in shaking-table tests of a 3-story building with non-post-tensioned rocking walls (Blomgren et al. 2018), showing a stable cyclic response at large drifts, damage concentrated at the intended replaceable device locations and repair feasibility.

The use of slip-friction connectors rather than vertical post-tension to control rocking has been extensively studied mainly at the University of Auckland, New Zealand, where these devices were first modeled as an alternative to hold-down connections for light-frame wood structures (Loo et al. 2012a). With this variation, some of the lateral drift increased while others decreased, but most values met the requirements. The end-chord tension decreased up to 45% and nail displacement up to 89%. The system was also modeled for CLT walls by Loo et al. (2012b) reporting that adding the slip-friction devices reduced base shear up to 85% and top accelerations up to 80%, while drift tended to increase but still meeting the requirements. This scheme was tested on LVL walls by Loo et al. (2014) including a shear key for resisting the lateral forces (Fig. 12c). The key was made of metallic pins inserted through the panels and bearing against vertical steel plates attached to the foundation. Hashemi et al. (2018b) presented an alternative shear key design consisting of an angle bracket with vertically slotted holes (wider at the top of the hole to accommodate rotations). The use of grooved friction plates combined with Belleville washers provided a restoring force to the connectors.

Loo et al. (2016) introduced a direct-displacement-based procedure applied to a 5-story CLT wall with slip-friction connectors and compared it with a traditional restrained wall, showing rocking benefits, increasing drift as a trade-off for smaller accelerations and base shears. Besides the slip-friction hold-downs, Hashemi et al. (2016) included slip-friction joints between panels to provide coupling and supplemental damping. Boundary steel frames were placed to resist gravity loads, with post-tension in the beam to self-center the structure. The system was modeled in a 6-story building in New Zealand, meeting design drift requirements and showing nearly no recognizable residual drift. Further numerical simulations of this system and tests of the connectors were presented by Hashemi et al. (2017). Hashemi et al. (2018a) analytically assessed the application of the system to multistory platform buildings by providing slotted bolted connections and a rotating pivot at the top of the walls to accommodate rotations. Additional tests and models have studied the out-of-plane (Valadbeigi et al. 2018) and rotational (Zarnani et al. 2018) behavior of the connectors. Jahnel and Cole (2017) conceptually proposed the use of vertical friction springs at the base of rocking timber walls. These devices provide a restoring force and an energy dissipation source that does not need to be replaced after an earthquake.

7 Future work

According to the present review, Fig. 13 shows the number of publications per year in recent decades related to SPTs in timber structures. Following Northridge and Kobe earthquakes, there has been an overall increase in research, especially in terms of isolation and supplemental damping, which has been maintained—or even increased—up to date. Rocking systems research shows a clear increasing trend since its introduction in timber engineering about 2005, and an especially strong tendency is observed upon the 2011 Christchurch Earthquake. Nowadays, rocking SPTs account for about 56% of the publications in the field.

The advantages of SPTs have been extensively demonstrated in the literature, with important reductions in accelerations, drifts and damage to structural components. However, technical and economic limitations of certain systems need to be addressed for wider implementations. For instance, many prototypes of supplemental damping in single-family light-framed construction used fluid viscous dampers, which usually is much more expensive than metallic or friction dampers. However, efficient utilization of the latter requires further research, as in general, the limited engagement (clearances) between these devices in the connections with the wood produces flexible interfaces that reduce the overall effectiveness of displacement-based damping. A possible solution is likely to encourage the inclusion of distinct materials or denser woods in the connection with dampers, as it was already presented in some of the recently reviewed publications. In addition, a limited range of displacement amplifying systems has been tested in timber in comparison to heavyweight supplemental damping of other materials- see, for example Mualla and Belev (2017), Baquero et al. (2016) and Symans et al. (2017)—and these may be adapted for lighter prototypes. Such amplification systems should benefit from the large yielding drifts of timber walls such that the forces needed at the dampers may be significantly reduced. Alternative amplification systems such as rotational devices may increase supplemental damping at smaller degrees of timber connection's yielding, which may increase the overall efficiency as well as preserve the structural integrity in case of a design level earthquake.

The low mass of light-framing construction suggests that the feasibility of base isolation relies on the use of sliders. However, this system faces the same challenge of supplemental damping regarding costs for widening its implementation in low- and mid-rise buildings, even when seismic isolation pricing has shown a steady reduction in prices during the last decades. On the other hand, most research on elastomeric seismic isolation dates back to the early 2000s, a time when mass timber construction and taller timber buildings were not as developed as today. However, rubber isolators may take advantage of these new massive construction types compared with light-framed construction (more massive and shorter periods). In both cases, developments should consider cost-effectiveness, and to this end, cheap sliders (Jünemann et al. 2009) or fiber-reinforced elastomeric isolators (Kelly 2002) may be an interesting option, as they are more economical than the steel-reinforced ones, and have been linked to massive systems such as masonry.

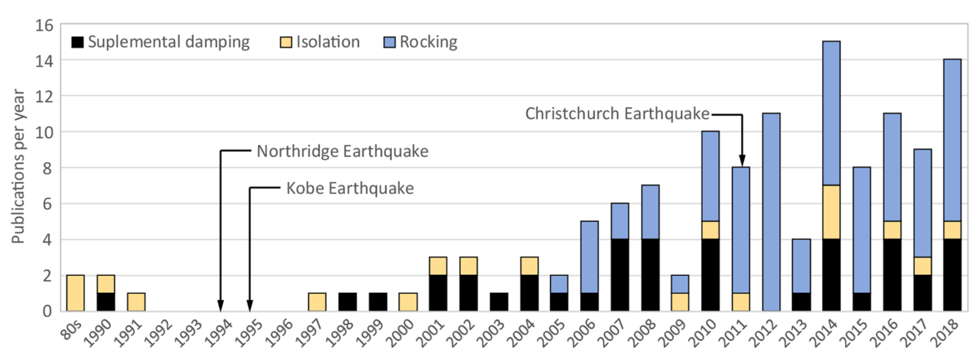

Fig. 13 Number of publications per year included in this review regarding SPTs utilization in timber construction

Rocking systems are expected to increasingly grow given the steady increment of mass (rigid) timber systems prone to rigid body rotation, especially oriented towards their use in walls with or without coupling dampers given its higher efficiency compared with frames. In this regard, recent research has also proposed cheaper dissipaters for twin walls (Schmidt and Blass 2017) which would contribute to widening their application. The dynamic behavior and interaction with beams and floors have already been characterized, but the response of non-structural components needs to be studied, especially for fragile components like drywalls, facades or windows. While several design procedures have been proposed in the literature for rocking systems, it is necessary to elaborate standard design guides including simplified analysis methodologies for expanding implementations. In addition, further research leading to decreased PT loading losses, and especially non-PT systems is expected, as efficiency in overall structural design directly correlates to the impairment of the PT system's overstrength.

8 Conclusion

Development of adequate Seismic Protection Technologies (SPTs) for timber structures has become a hotspot for timber engineering research since the losses experienced in the last major earthquakes. Investigations in this field have shifted from a rather application of concrete and steel technologies to an identification of the implications and limitations of SPTs' usage in timber buildings, enabling thus the development of effective systems capable of reducing seismic demands up to 90% and above. Still, several issues need to be further developed, especially in terms of reducing prices (accordingly to the lower seismic loading of timber structures), rate effects, and efficiency after cyclic loading.

Supplemental damping has been a recursive solution in light-framing that strongly profits from the large elastoplastic deformation capacity of light timber assemblies. With the right amplification system, efficiencies of supplemental dissipation may reach new standards of performance unseen in steel and concrete buildings. In addition, this technique has been very effective in reducing drifts while adding stiffness—two very valuable outputs for light timber construction. Some of the last advances have also shown many opportunities in taking advantage from the relative displacement of hybrid timber-steel and timber-concrete buildings, so applications in-between gravitational and lateral force resisting systems of mid- to high-rise timber buildings may be increasingly exploited in the future, which also applies to rocking systems.

Base isolation systems have been tested in a wide range of timber buildings. The efficiency of elastomeric isolation may be an option for future mass timber construction. For light-framing however, sliding systems seem to be a much better option from the technical standpoint. In the opinion of the authors, the future of isolation in timber buildings clearly needs to find cheaper solutions compared to steel and concrete structures, which given the smaller loading of the timber and ease for prefabrication, is entirely feasible. Isolation has also shown drift reductions up to 90% along with comparable drops of lateral forces and accelerations, so it is clear for laterally constrained designs that cheap isolation systems can be very beneficial not only from the technical but from the economical point of view.

Rocking systems have shown very efficient performance in mass timber wall constructions because they take advantage from mass walls' inherent propensity for rigid body rotation as well as their high specific stiffness—thus elevated self-centering capabilities. In addition, the rotational as well as elastic timber deformation can be fully exploited for even larger supplemental damping dissipations which turns to be especially beneficial for wall assemblies with scarce connections (and thus dissipation capabilities) as those built from mass timber walls. As in the case of sole supplemental damping, ease for design guidelines is greatly needed under consideration of all the complexity encountered in design, not only in terms of instant but long-term performance. To this end, clear definitions for the calculation of overstrength for timber assemblies—especially light-framed and mass timber walls—as well as detailing of connections to the gravity system is greatly needed in codes. Some investigations showed the technical feasibility to implement rocking systems in multi-story buildings as tall as 14 stories, creating thus new opportunities and challenges for timber construction in highly prone seismic areas. In conclusion, the authors believe that even when timber has traditionally been regarded as one of the best seismic construction materials, it has the potential for fully exploiting SPTs into very high standards of cost-effectiveness and structural efficiency, so this potential should be further researched for timber to become the best possible choice for many applications in highly seismic zones.

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