Chapter 9

Ductile Behavior of Timber Structures under Strong Dynamic Loads

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Additional information is available at the end of the chapter

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Abstract

Due to their comparatively low mass that implies reduced horizontal dynamic loads even during strong earthquakes, wood-made buildings might be a good choice in seismic prone regions. To meet the modern design philosophy requirements, however, such structures should be able to behave in a ductile way under exceptional events. By presenting a brief review of the latest developments in the field, this chapter investigates on when and to what extent historical and modern timber buildings may exhibit a ductile and dissipative behavior. A special focus is given to the crucial role of connections and to the difficulties involved by their mechanical model when carrying out code-based non-linear dynamic analyses. Although a ductile behavior is typically required under strong earthquakes, it is to note that a well-designed ductile structure may also be able to withstand other exceptional events as, for instance, tornadoes or blasts.

Keywords: ductile behavior, strong dynamic loads, timber structures, seismic performance

1. Introduction

To be able to exhibit a ductile behavior is a bonus that a structure can exploit when exposed to extreme and unpredictable actions like those applied by earthquakes, hurricanes, snowstorms, fire or explosions. In fact, it would be uneconomical to design structures that withstand such infrequent events still deforming in the elastic range. In addition to cost-effective reasons, a significant advantage of the ductile behavior is the great amounts of input energy that can be dissipated during the plastic motion, considerably higher than what could be dissipated in the elastic range. Further advantages are the beneficial redistribution of stress within statically undetermined systems and the warning that large deformations may give to inhabitants in case of exceptional loads. Aimed by these reasons, the seismic codes of practice typically require that buildings survive earthquakes of moderate severity deforming in the elastic range whilst they behave in a dissipative ductile way, even getting damaged severely but without collapsing, during strong seismic events. Local curvature ductility and strength hierarchy requirements ensure such extreme performance of buildings with the final goal of life safety.

Based on this well-established design philosophy, a lot of experimental data, numerical procedures and code provisions are in fact available to design earthquake-resistant buildings made by steel or reinforced concrete [1–3]. On the contrary, a lack of sufficient research results and detailed code requirements still affect the design of wood structures in high seismicity areas, despite the growing interest in natural and renewable materials. Light weight as they typically are timber structures might possibly resist earthquakes better than other kind of structures. Being proportional to the building masses, in fact, the horizontal loads acting on light structures during a given earthquake are lower than those acting on heavier structures, so that reducing the building active mass may be even a strategy for seismic stress control [4, 5].

On the other hand, to meet the modern design philosophy, timber structures are expected to exhibit a ductile and dissipative behavior—besides adequate strength and stiffness—under strong earthquakes. Ductility is in fact the capability to undergo large inelastic displacements without significant reduction in strength. On the other hand, high values of ductility usually imply large amount of energy dissipated during the dynamic motion, being the energy dissipated in each cycle related to the hysteresis area of the load-displacement diagram.

With the aim of investigating on the dynamic behavior of timber structures under extreme conditions, this survey focuses on the post-elastic behavior of wood constructions. The chapter is composed of two main parts. The first one deals with the ductile behavior of historical buildings (Section 2). A brief review on the performance of existing wood-made buildings under past events is presented in Section 2.1 while experimental data on the ductile and dissipative behavior of ancient timber buildings, as available from the literature, are provided in Section 2.2. The remarks listed in Section 2.3 close the first part of the chapter. The second part refers to the ductile and dissipative behavior of modern timber constructions (Section 3). The key role of metallic connections is addressed in Section 3.1, whereas some practical aspects to be taken into account before carrying out a code-based non-linear dynamic analysis of a timber structure are highlighted in Section 3.2. Some brief conclusive notes are provided in section 3.3.

2. Ductile and dissipative behavior of ancient timber buildings

Until the advent of reinforced concrete, wood buildings have been popular for centuries in many seismic areas like China, Japan, Greece, Turkey and Balkan countries. In other geographical areas prone to earthquakes or tornados (North America), wood-frame single-family houses and low-rise multi-family dwellings never stopped to be built since colonial times. Timber constructions are today still very common in countries where wood is in good supply (USA, Canada, North Europe and New Zealand), but a growing interest is developing even in regions where adobe and masonry have been the most typical construction materials for centuries (e.g. Mediterranean countries). Assessing the performance of timber structures during past severe natural events may be thus of great interest not only to preserve the architectural heritage but also to improve the design of new buildings.

2.1. Performance of timber buildings during past natural events

Somehow unexpected, the finding that many ancient wooden constructions remained nearly intact after strong earthquakes, whilst modern reinforced concrete buildings collapsed, as emblematically shown in **Figure 1**, gave new impetus to build timber constructions even in seismic regions. Several authors documented, in fact, a generally good performance of timber buildings during natural events as earthquakes, hurricanes, tornadoes and snowstorms [6–12].

Actually, most of the existing timber constructions were built according to conventional guidelines based on the empirical knowledge of the past while only the more recent ones (a little part) are engineered structures meeting modern anti-seismic standards. Mainly based on inspections made after natural disasters, best-practice rules were suggested—although not always applied—in the past [13]. It may be interesting to mention that the first European anti-seismic recommendations, which were enacted in 1783, after a devastating earthquake struck Calabria and Sicily (at that time belonging to the Borbone Kingdom), prescribed to strengthen buildings by means of wood frames in filled in the masonry walls. The timber frames were recommended to be suitably connected to each other's, to the ground and to the floors, see **Figure 2** [14]. Even more interesting is the fact that such an aseismic system (adopted in Italy till the twentieth century and referred to as "*casa baraccata*"), had actually been borrowed from the traditional little wooden single-story houses (called *baracche*) which were usually built near to the palaces of nobles to be used as earthquake-proof shelters [15–17]. Although not written, aseismic recommendations similar to—and most likely inspiring—the Borbone's ones were adopted in Lisbon a few years before, after the catastrophic earthquake and the ensuing tsunami that struck the town in 1755. Based on the knowledge collected from survived buildings, the *Pombalino* constructive system (imposed by the Prime Minister Marquis of Pombal) consisted in fact of a three-dimensional timber structure (called *gaiola* due to its cage-like structure) included in the masonry walls above the first floor of the building, see **Figure 3** [18–20].

Figure 1. Different damaged conditions of traditional wooden and reinforced concrete buildings after the 1999 Duzce earthquake. Image taken from [6].

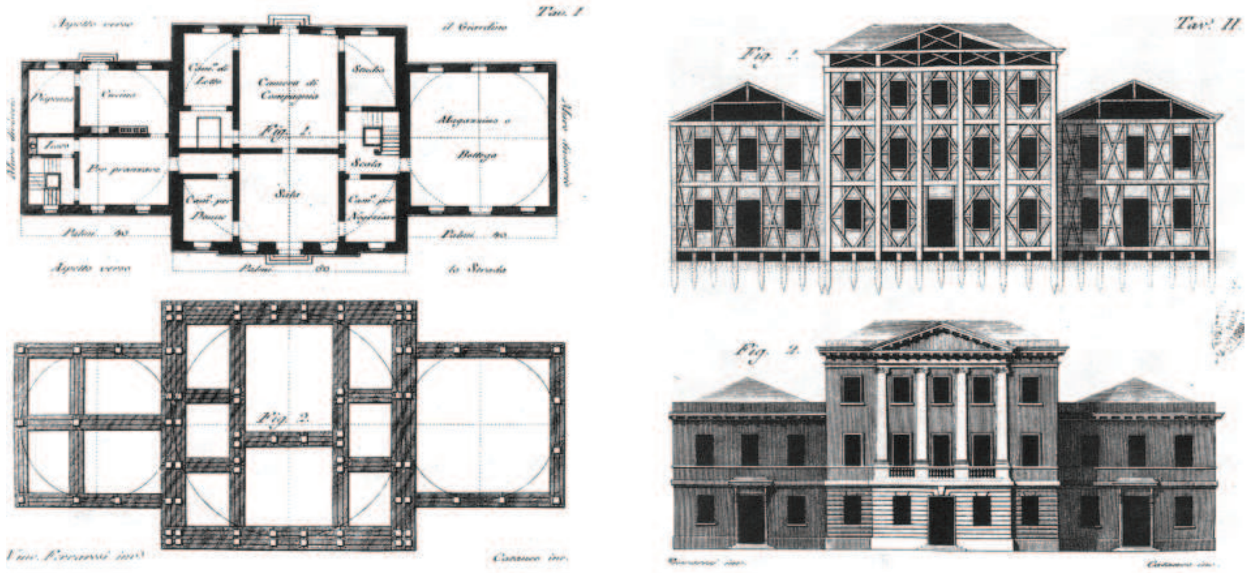

Figure 2. First European aseismic recommendations dating back to 1783 [14].

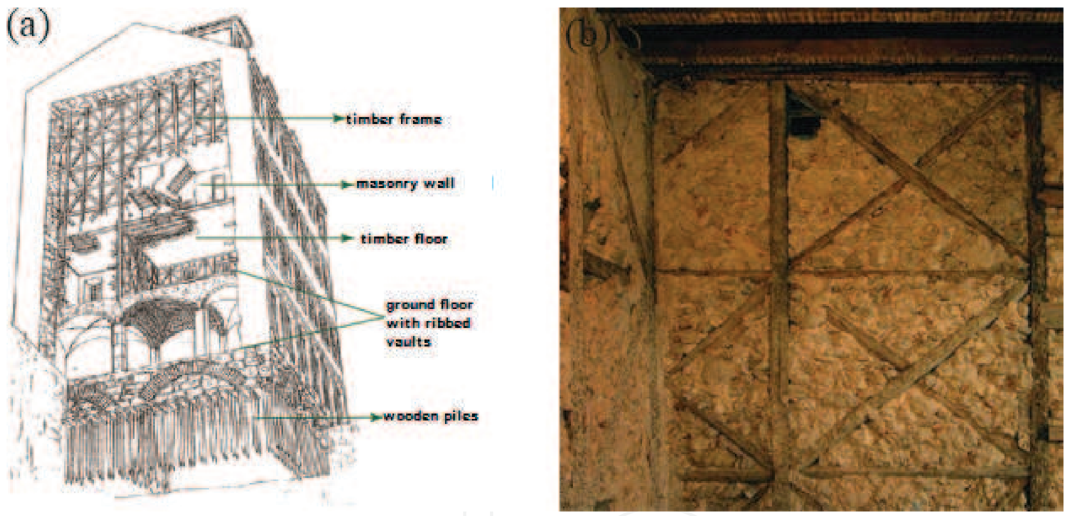

Figure 3. (a) Pombalino building scheme [18]; (b) Pombalino building walls.

Due to the different role given to the timber structure, the "Borbone" and the "Pombalino" constructive systems were quite dissimilar in their mechanical behavior, as discussed in Ref. [16]. Both of them are, however, examples of ancient timber-framed buildings. Although the concept of structural ductility was still very far to be introduced in building design, the role of timber frames in traditional constructions was in fact that of withstanding lateral loads by allowing large—and usually inelastic—displacements (like a sort of reinforcement skeleton), a role which cannot be played by unreinforced masonry.

Constructive systems adopting timber frames with infill masonry (or adobe or stone) may be actually found in different historical periods and geographical areas, starting from one of the most ancient examples still visible: a Roman house of the first century BC in the archeological site of Herculaneum (Italy) (**Figure 4a**). More recent examples are the several well-preserved

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Figure 4. Historical timber-framed masonry buildings: (a) house wall from Herculanum (opus craticium attested by Vitruvius in the first century BC); (b) nineteenth-century Ottoman house; (c) sixteenth-century timber-frame house in Paris (France); (d) St. Bartholomew's Gatehouse built in London in 1595 and survived to the 1665 great fire; (e) German typical timber-framed houses (Fachwerkhäuser); (f) Anne Hvide's House built in 1560, Denmark.

Ottoman traditional timber-frame houses (see **Figure 4b**), that may be still found in earthquake-prone areas as Turkey, Balkan countries, Iraq, Siria and Egipt [6, 9, 21]. Likewise the *Pombalino* system, also in the Ottoman houses, the timber frames were introduced from the first floor up (for this reason they are also referred to as half-timbered frames). Further examples of timber-framed buildings (dating back even to the medieval period) may be also found in European countries as Germany, the United Kingdom, Greece, France, Spain, Poland (see e.g. the examples given in **Figure 4c–f**), as well as in India [22, 23], the USA [24], China [25] and Peru [26]. An overview of the history of timber framed buildings may be found for instance in Ref. [32].

Different from the timber-framed systems, other wooden ancient buildings can be also broadly found (see **Figure 5**).

The widespread use of wood as structural material even in earthquake-prone areas highlights the awareness they had in the past of the good performance of such a material even under extreme conditions. Of course, mistakes and experience taught how to build timber structures to better withstand strong dynamic actions. The following shortcomings were typically found, in fact, to be responsible for the inefficient structural behavior of wood buildings in past natural disasters: lack of adequate foundation anchorages or inadequate foundations soil; presence of a soft-story mechanism (typically due to large openings at the ground floor of multi-story buildings); insufficient connections between major components or lacking lateral bracing, as

Figure 5. Ancient wood constructions: (a) Hōryū-ji Pagoda (Japan, 607 A.C.); (b) Kizhi Pogost Church (Russia, seventeenth century); (c) Fogong Temple Pagoda (China, 1056 AC); (d) Heddal stave church (Norvey, thirteenth century).

documented, among others, in Refs. [6–11]. Other causes of failure were also detected as, for instance, failure of infill materials in timber-framed buildings, failure of chimneys, collapse of neighbor buildings, structural changes or excessive loadings given at a certain stage of the building life which caused very large inertia forces [6, 9, 11, 27, 28]. However, the main lesson drown from past events was that, to better resist to lateral loads, a timber structure should be regular, have little openings, possess effective connections, bracing elements and ground anchorages. In other words, it should eventually behave as a box-like unit.

2.2. Ductility of historical buildings

Apart from the qualitative considerations recalled in the previous section, we can ask whether and to what extent existing (and often very ancient) timber structures can be able to exhibit a ductile and dissipative behavior under extreme conditions. Unfortunately, answering this question is not an easy task, since the actual dynamic behavior of historical timber buildings is hardly predictable and it is usually documented, as far as for the past, only by empirical data. The task is even made more difficult by the different constructive techniques, wood species, kind and quality of infill materials and by the large variety of joints adopted in ancient structures. Some recent studies can be, however, exploited to have some clues.

2.2.1. Timber-framed buildings

Experimental researches were carried out to assess the behavior of the *Borbone* constructive system, [16, 17, 29]. Particularly remarkable are the results provided in Ref. [16], relevant to in-plane cycling tests performed on two full-scale models of *Borbone* walls, following the UNI EN 12512:2006 protocol [30]. Experimental findings evidenced a non-linear behavior of specimens with comparatively high values of ductility. The latter was calculated as the ratio between the maximum displacement $u_{\max}$ and the displacement at yield u_y, namely

$$\mu = \frac{u_{\max}}{u_y} \tag{1}$$

The quantity defined by Eq. (1) is sometimes also referred to as the static ductility. To quantify the dissipation of energy in the plastic range, the hysteresis equivalent damping ratio was calculated in Ref. [16], as defined by

$$\xi_{eq} = \frac{1}{2\pi}\frac{E_d}{E_p} \tag{2}$$

Based on the Jacobsen approach [31], Eq. (2) considers the ratio between the energy dissipated for a half hysteresis cycle, namely E_d, and the potential energy E_p that an equivalent simple oscillator would store for the same displacement achieved in the hysteretic half-cycle (see **Figure 6**). The value of ξ_{eq} was evaluated in Ref. [16] for the third cycle of each level of displacement imposed, as suggested by the UNI EN 12512:2006 protocol [30]. The cumulative dissipated energy E_{d_cum}, obtained by summing the loop areas of successive cycles, was also calculated. The values of μ, E_{d_cum} and ξ_{eq} obtained in Ref. [16] for infilled and bare walls are provided in **Table 1**.

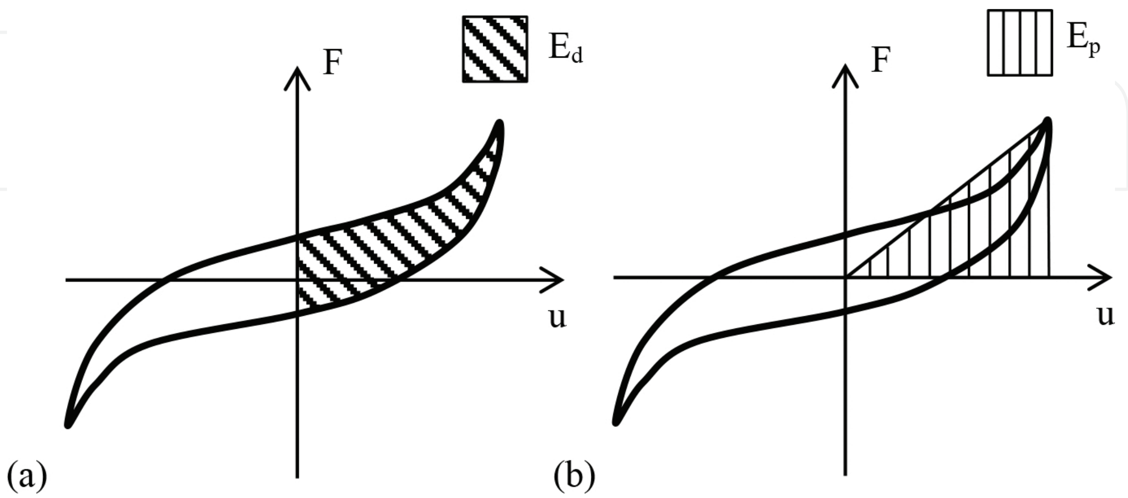

Figure 6. Pinched hysteresis loop (third cycle): (a) dissipated energy for a half cycle; (b) potential energy of the equivalent oscillator for a half cycle.

Test specimens	Frame	μ	E_{d_cum} (kJ)	ξ_{eq} (%)	Drift (%)
Real-size *Borbone* walls [17]	Infilled	7.6	19	6–8.9	2–2.6
	Bare	1.36	1.6	1–23	0.15–2.71
Real-size *Pombalino* walls [32]	Infilled	5.48–17.95	2–2.8	15	1.76–3.34
	Bare	11.6–15.28	0.75–2.8	11–15	2.55–2.61
Real-size Ottoman walls [21]	Infilled	n.a.	1.8–11	7.80	4–6.5
	Bare	n.a.	0.6–8	7.79	3.5–8.7
Real-size *dhajji-dewari* walls [23]	Infilled	3.7	n.a.	5–25	4.2–5
	Bare	3	n.a.	7	4.2–5
Half-scale *quincha* walls [26]	Infilled	6.2	n.a.	n.a.	n.a.
	Bare	15.4	n.a.	n.a.	n.a.

Note: n.a., not available.

Table 1. Ductility ratio and dissipation capacity of historical timber-framed walls.

In the infilled specimens, the energy dissipation was found to depend mostly on friction and small ruptures in the masonry. In bare specimens, the almost wholeness dissipation was found instead to be concentrated in the connections, where two mechanisms occur contemporarily: the first one is related to the compression and crushing of wood grain with the formation of a cavity due to the presence of nails (which is also responsible for the pinching effects discussed in the following); the second one concerns the inelastic deformation of the iron nails, [16, 17].

The dynamic behavior of the *Pombalino* timber-framed buildings in Lisbon was investigated through experimental tests on specimens taken from actual sites [28] or rebuilt in laboratory [19, 27, 32]. All of them assessed a rather good ductile behavior of *Pombalino* walls. In particular, the results given in Ref. [19], relevant to real-size reconstructions of *Pombalino* walls under cyclic displacement sequences following the CUREE protocol [33], showed fat hysteresis loops dissipating reasonable amounts of energy (although the actual values are not given by the authors). The ductility ratio was found to be around 3. Significant pinching effects, related to gaps and residual displacements which form in the connection zones and between masonry and timber members were detected. A rather accurate non-linear hysteresis model was also developed in the same paper to capture the dynamic response of the walls. Similar experimental tests on real-size laboratory reconstructions of infilled and bare-framed *Pombalino* walls were carried out in Ref. [32]. The values of μ, E_{d_cum} and ξ_{eq} found in Ref. [32] are collected in **Table 1**.

Experimental tests on eight different timber frames with and without infill (brick and adobe) or cladding, all reproducing portions of the typical *himis*-timber-framed Ottoman constructions, were carried out in Ref. [21]. The force-displacement curves obtained for reversed cyclic in-plane lateral loading showed stable hysteretic loops with highly dissipative behavior. The authors suggested to evaluate the cumulative energy dissipation capacity through the following empirical formulas [21]:

$$E_{d\_cumW} = (0.873W_{eff} + 0.055A_{eff})(1.1865D_{\max} + 0.5697)^{3/2} \tag{3}$$

$$E_{d\_cumWO} = (W_{eff} + 0.0638A_{eff})(0.19D_{\max} + 1.0469)^{2} \tag{4}$$

Here E_{d_cumW} and E_{d_cumWO} denote the cumulative energy dissipation capacity (expressed in Joule) of specimens with and without infill/cladding, respectively; while W_{eff}, A_{eff} and $D_{\max}$ are the effective width (m), the effective area (m^2) and the maximum lateral top displacement (mm) of each framed wall considered in the experimental tests. The values of E_{d_cum} as obtained by applying Eqs. (3) and (4) and of ξ_{eq}, as defined by Eq. (2), were derived from the data given in Ref. [21] and are provided in **Table 1** both for infilled and bare frames. A high ductile behavior was found to be exhibited by the considered timber-framed walls, although the actual values of the ductility ratio μ were not provided in Ref. [21]. The highly dissipative behavior detected for Ottoman walls was found to depend on the dissipative plastic behavior of nailed connections, as also confirmed by other authors [22].

The results of in-plane quasi-static cyclic tests on full-scale *dhajji-dewari* walls (wooden-braced frame system with masonry infill, typical of India and Pakistan) were given in Ref. [23]. The tests highlighted a very strong resilience of the *dhajji-dewari* system against lateral loads, which is almost totally due to the performance of the timber framework. Also in this case, a rather good ductile and dissipative behavior is found in the experimental tests. The values of μ and ξ_{eq} obtained in Ref. [23] for infilled and bare walls are given in **Table 1**.

Finally, the data taken from [26] relevant to experimental tests carried out on half-scale *quincha* walls are also provided in **Table 1**. (*Quincha* is a Peruvian traditional timber-framed constructive system consisting of an adobe-made ground floor and upper stories built with timber frames infilled with a weave of canes and mud).

A qualitative diagram is provided in **Figure 7** showing some of the typical features evidenced by the experimental cyclic tests on timber-framed walls, infilled with masonry and connected with metallic nails. The main features evidenced in Figure 7 are (a) non-linear behavior; (b) almost symmetric curves; (c) indistinct yield point; (d) lateral stiffness degradation for increasing loading cycles; (e) pinching effect after the first load cycle; (f) strength degradation at the same deformation level for increasing loading cycles; (g) strength degradation for higher deformation;(h) fat hysteresis loops which implies large amounts of energy dissipated (narrowed loop areas are generally found, however, for successive load cycles or for higher deformation levels); (i) rather high values of ductility.

The so-called "pinching effect" is due to the formation of a cavity around the fasteners due to irrecoverable crushing of wood (this effect occurs after the first loading cycle) which implies a reduced stiffness at load reversals (the connection stiffness is due, in this phase, to the sole contribution of metallic fasteners). As soon as the contact with the surrounding wood is reestablished, at increased deformation levels, the stiffness rapidly increases which leads to the typical pinched shape of curves in the load-displacement diagram. It can be noted, finally, that features very similar to those illustrated in **Figure 7** and discussed before were

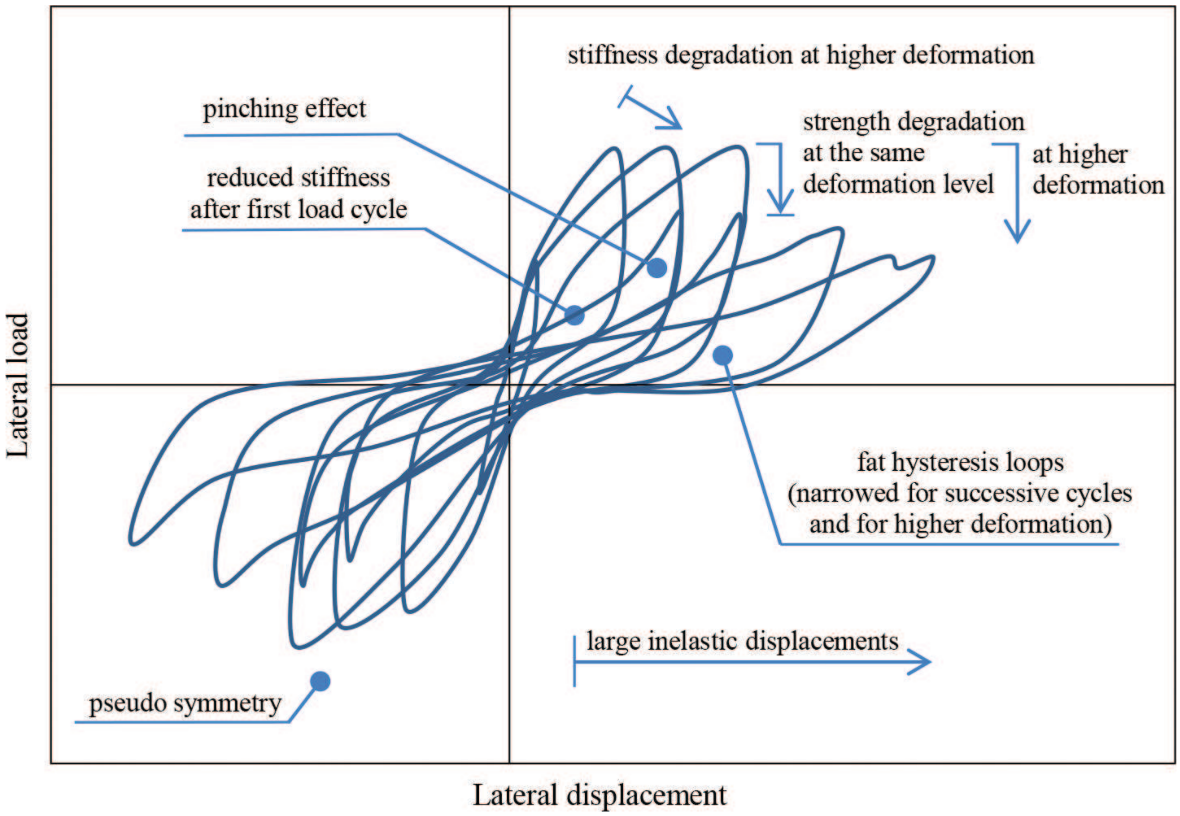

Figure 7. Qualitative hysteresis loops as obtained from experimental tests on timber framed walls.

also detected for modern timber walls (plywood shear, strand-board diaphragms), as can be inferred by examining, for instance, the experimental load-displacement curves provided in Ref. [34].

2.2.2. Other kinds of ancient wooden constructions

Experimental studies were carried out to assess the dynamic behavior of traditional Chinese [35–37] and Japanese [38–40] constructions. Of particular interest to the purpose of the present study are the results provided in Ref. [37]. They are relevant to scaled specimens, reproducing a prototype of a historical Chinese timber palace, subjected to cyclic lateral loading. Stable and large-area hysteretic loops were found in the tests, associated with rather high values of ductility (9–19) and energy dissipation (not quantified). The equivalent viscous damping was evaluated to reach even values of 20–30%. It is worth noting that Chinese and Japanese constructions were found able to exhibit a rather high ductile response and to dissipate large amount of energy, despite they typically avoid metallic connections. Energy dissipation was found to be due, in fact, to friction and embedding between structural elements at the contact interfaces inside the wooden joints.

Further experimental studies can be also quoted, for example [41], confirming a rather good ductile and dissipative behavior of ancient timber constructions.

2.3. Some remarks on the ductility of ancient timber constructions

Before closing the present section, the following remarks can be drawn.

i. Ductile behavior and energy dissipation are two key points of the modern aseismic strategy adopted by current standards. Experimental findings showed that historical timber buildings can generally be able to meet these modern requirements.

ii. **Table 1** substantiates statement (i). With reference to the classification given by EC8 [1], recalled in **Table 2**, three kinds of the traditional timber structures referred to in **Table 1** may be assigned to the high ductility class (DCH), namely the *Borbone*, the *Pombalino* and the *quincha* walls (all in the case of infilled timber frames). A low ductility class (DCL) should be assigned instead to the *dhajji-dewari* walls. The class of ductility of the Ottoman traditional walls cannot be assessed through the available data.

iii. Although giving an interesting portrait of the ductile and dissipative capacity of traditional timber buildings, the data collected in **Table 1** should be compared with caution due to the different settings (laboratory set-ups, loading procedures, recorded data analysis, yield and ultimate deformation evaluation) adopted in the experimental studies mentioned. For instance, the total amount of energy dissipated can be strongly dependent on the load protocol, which is usually different from an experimental research to the other. In addition, it can be noted that a different ductile behavior was sometimes detected in experimental tests from positive to negative direction of load, although such an aspect has not been evidenced in **Table 1** for the sake of brevity.

iv. The last column of **Table 1** reports the values of the drift (in %) as given by the ratio between the maximum lateral displacement and the wall height. This is in fact a very significant parameter to be considered when large displacements are involved.

v. Despite the extreme attention paid by the authors in reproducing, as faithfully as possible, the in-situ conditions(timber species, infill materials, geometry, ground constraints, connections, and so on), the results obtained in laboratory on rebuilt models of parts of ancient constructions should be obviously used with great care to predict the actual dynamic behavior of existing whole buildings, also in view of all the aspects affecting the behavior of real structures (material degradation, efficiency of connections, internal damage, workmanship irregularities, tridimensional behavior of the building).

vi. A crucial role in the ductile behavior of ancient timber structures was found to be played by connections, since timber elements generally do not exploit their latest strength resources [21, 42]. Based on this statement, some retrofitting solutions were also proposed to improve the dynamic behavior of ancient structures under dynamic loads [27, 42]

3. Ductile and dissipative behavior of modern timber structures

Besides the more common single-family and low-rise houses, spectacular and daring-shaped modern timber buildings may be even encountered nowadays in many countries, as the few

μ	Class of ductility	q	Structural type
≤4	DCL Low capacity to dissipate energy	1.5	Cantilevers; beams; arches with pinned joints; trusses joined with connectors; mixed structures consisting of timber framing and non-load bearing infill.
4<μ<6	DCM Medium capacity to dissipate energy	2	Glued wall panels with glued diaphragms connected with nails and bolts; trusses with doweled and bolted joints.
		2.5	Hyperstatic portal frames with doweled and bolted joints possessing medium capacity of rotational ductility.
μ≥6	DCH High capacity to dissipate energy	3	Nailed wall panels with glued diaphragms, connected with nails and bolts; trusses with nailed joints
		4	Hyperstatic portal frames with doweled and bolted joints possessing high capacity of rotational ductility.
		5	Nailed wall panels with nailed diaphragms, connected with nails and bolts.

Note: The q-values should be reduced by 20% if the building is non-regular in elevation.

Table 2. Ductility classes and behavior factors q for different typologies of timber structures, according to EC8 [1].

instances of **Figure 8** let imagine. A feeling for eco-friendly and renewable materials, together with the easy of production and transportation from the past, adds new motivations to the construction of wooden buildings.

As discussed in the introductive section of this chapter, modern structures are required to be ductile and dissipative, particularly when they are built in seismic areas. While timber structures are uniquely recognized to be able to meet such requirements provided that they are

Figure 8. (a) Dolomites, Italy; (b) Vancouver, Canada; (c) London, Great Britain; (d) Odate Stadium, Japan.

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regular, hyperstatic and connected with ductile fasteners (as also confirmed by **Table 2**), most of the issues related to evaluating and modeling this ability are still under discussion.

3.1. Crucial role of connections

Connections in modern timber buildings are metallic devices ensuring transmission of forces between structural elements. Their design is the most strategic part of the structural project of a timber construction, since from the characteristics of the connections (type, mechanical properties, geometry, spacing, assembly techniques) may strongly depend the stiffness, the strength, the ductility and the energy dissipation of the whole structure.

Although some constructive typologies (such as moment-resisting timber frame systems, timber shear panel systems and cross-laminated panel systems) are indicated as being particularly able to ensure a ductile behavior under extreme dynamic lateral loads [43], it is the connection design that eventually decides the ductility resources of a timber structure. The same structural type may be, in fact, assigned to different ductility classes in dependence of the rotational ductility capacity of its connections, as can be inferred, for instance, by the classification done by EC8, recalled in **Table 2**.

The most common connections in modern timber structures are the dowel-type mechanical fasteners (nails, screws, dowels, bolts, rivets) which deeply penetrate into the wood to transfer the load by means of wood bearing and connector bending. Dowel-type connectors can be used alone or in combination with metal predrilled plates. Joints with dowel-type fasteners are expected to be ductile due to the highly nonlinear behavior of the wood under embedding stresses and the plastic behavior of the steel fasteners in bending [44]. Nevertheless, they can sometimes be affected by sudden and brittle failures like block shear or splitting [45]. Ten different types of failures (six in single shear and four in double shear) are considered by the European standards for dowel-type timber connections [46].

As a matter of fact, timber members and metallic joints play different roles in the seismic behavior of timber structures. Since the failure mechanisms of wooden elements are mostly brittle, the timber members are required to remain in the elastic range even under very strong events. The task of satisfying the demand of ductility is entrusted instead to the metallic connections which are expected to sustain large inelastic deformations while preventing collapse. The ductile behavior of connections is influenced both by metallic fasteners (which may behave in a ductile or brittle way depending on whether plasticization is attained or not) and by the strength properties of the wood surrounding the connection zone (direction of the grain with respect to the load direction).

Preventing brittle failure may guarantee an adequate ductility to the whole structure. Complying some strength hierarchy rules can assure a ductile behavior to timber structures. In particular, it is essential to design the fasteners to be weaker than the wood members they are connecting, so that they can yield and dissipate great amount of energy. On the other hand, the weaker the fasteners, the lower their bearing capacity. A way of ensuring both adequate ductility and sufficient bearing area is using a large number of weak fasteners. Some alternatives to improve the performance of dowel-type joints are discussed in Ref. [47].

Although the plastic properties of the steel fasteners alone are well-known and their behavior under cyclic loads easy predictable, the non-linear response of the assembly of metallic connectors and surrounding wood is rather difficult to predict, since it is not a cross-section property (as for reinforced concrete). In fact, the behavior of the timber connections depends from several factors, some well-known as the strength properties and the geometric configuration of involved materials, others affected by uncertainty as the influence of neighboring metallic fasteners or the interaction between fasteners and surrounding wood. This makes rather difficult to develop an analytical model able to reproduce the behavior of a timber connection.

Most of the features evidenced in **Figure 7** and discussed in Section 2.2.1 characterize the behavior of metallic timber connections, as can be inferred from **Figures 9a** and **9b**, which provide qualitative examples of the typical hysteretic behavior of riveted and nailed connections, respectively. In particular, two phenomena were found to be typical of the hysteretic response of steel dowel-type connections, as recalled in Ref. [43]. The first one is the *pinching effect* implying different hysteretic curves from the first to the subsequent load cycles (see **Figure 9**). The second one, referred to as the *memory of material*, is due to a dependence of the load-slip curve from the loading history. Both these phenomena may influence the ductile behavior of a timber structure.

3.1.1. *Influence of the pinching effect on the ductile behavior of connections*

The *pinching effect* is a very typical feature of the hysteretic behavior of dowel-type connections affecting both historical and modern timber constructions. The mechanical causes of it have been discussed in Section 2.2.1. This effect has been documented by many authors, as for instance [48–52]. It was found, in particular, that for a given displacement level, the highest resistance and widest hysteresis loop was attained at the first load cycle, whilst the subsequent cycles were narrowed and achieved lower resistance, stabilizing after about three cycles (see **Figures 9a** and **9b**). Stabilization of the pinched curve after three cycles is also referred to in UNI EN 12512:2006 [30]. Due to the reduction of the area of the hysteresis loop, the *pinching effect* may be actually responsible for a reduced amount of energy dissipation, although connections are still able to exhibit high values of ductility.

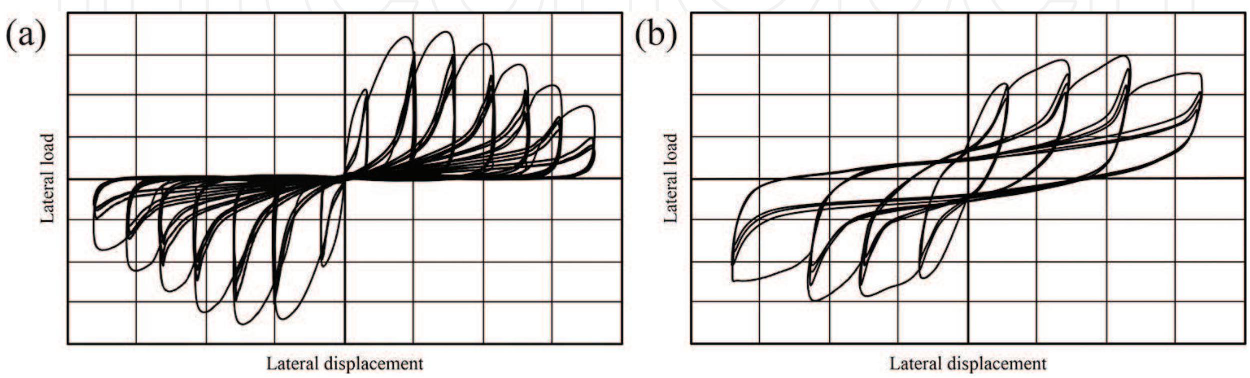

Figure 9. Typical hysteretic curves from cyclic tests on metallic (a) riveted connections and (b) nailed connections.

In modeling the mechanical behavior of a steel dowel-type connection for numerical analysis purposes, the pinching effect should be considered. A discussion on how this can be done may be found in Ref. [34], even if standard models comprehensive of the pinching effect and of the strength and stiffness degradation are not yet available, neither suggested by codes of practice.

3.1.2. Influence of the load history on the ductile behavior of connections

From the results available in the literature, it is clear that the hysteretic behavior of timber connections may strongly depend on the type of experimental test carried out (dynamic, static, cyclic, monotonic) as well as on the testing protocol adopted. On the other hand, while different protocols are available to carry out cyclic loading tests on timber structures, for example, EN 12512 [30], CUREE-Caltech standard [33], UBC protocol [11], a consensus on the best protocol to be assumed as standard has not been yet reached [48]. Many experimental findings proved, however, the influence of the load history on the final results.

It was shown in Ref. [48] that a connection usually reaches its maximum load at a lower deformation under cyclic loads than under monotonic loading. In Ref. [50], it was found that the ductility ratio of timber shear walls may be much higher when measured under static monotonic tests than when measured under dynamic tests. These experimental findings indicate that results from monotonic tests tend to overestimate the load-deformation behavior of connections with respect to cyclic loading tests, and therefore should be avoided when determining the seismic performance of timber buildings [48]. Dynamic tests are surely the best choice to capture the behavior of timber structures under seismic or wind loads, also in view of the fact that the failure modes may be very different in static and dynamic conditions [50]. The hysteresis loops obtained from dynamic tests were, however, found to be very sensitive to the protocol adopted [11, 53].

The dependence of the connection ductility from the experimental test may be also inferred from **Table 3**, where are collected the ductility ratios experimentally obtained for different timber connections [44, 48, 51–52, 54]. **Table 3** may be quite convenient to have an idea of the ductile capacity of timber connections, although the data herein provided should be compared with care, owing to the different specimens, test setups and loading protocols involved in the tests (the reader is referred to the papers quoted in the table for any detail).

Analogously, the ductility ratios of modern timber walls are given in **Table 4**, as derived from Refs. [50, 55, 56]. The data collected in **Table 4** highlight the good ductility which may be exhibited by modern timber constructions, although comparing the data collected in **Table 4** needs again caution. It can be also noted, finally, that the hysteresis curves obtained by testing modern timber walls with nailed connections evidenced features similar to those of **Figure 7**, as can be inferred for instance from diagrams provided in Ref. [50–51, 55, 57].

3.2. Non-linear dynamic analysis to predict the seismic response of timber structures

The non-linear time-history analysis (NLTHA) is the most comprehensive procedure allowed by seismic codes to design earthquake-resistant structures. It involves a complete time-history investigation under different spectrum-compatible ground motions. Despite its potential, the

Connection type	Wood elements	Loading	μ
Steel plates with bolts [48]	Glulam members	Monotonic	3–4.8
		Cyclic	2.53–2.91
Steel plates with glulam rivets [48]	Glulam members	Monotonic	16.4–20.4
		Cyclic	10.74–15.96
Steel brackets with nails or screws [51, 52]	XLam panels	Cyclic (parallel to grain)	3.01–6.36
		Cyclic (perpendicular to grain)	3.82–4.83
Dowel-type [44]	XLam members	Cyclic	1.3–2.1
Dowel-type reinforced with self-tapping screws [44]		Cyclic	3.4–7.3
Slotted-in steel plates with nails [54]	glulam members	Monotonic (parallel to grain)	11.9–31.9

Note: XLam, cross-laminated.

Table 3. Ductility of connections as obtained from experimental tests.

Test specimens	Connections	Loading	μ
Shear walls sheathed with plywood [50]	Plates to stud nailing	Monotonic	14
		cyclic	9.3
Shear walls sheathed with OSB [50]	Plates to stud nailing	Monotonic	13.2
		cyclic	7.7
Cross-laminated walls [55]	Hold-downs and brackets with nails, screws and rivets	Cyclic	3.65–7.54
Shear walls sheathed with OSB [56]	Nailed steel brackets and hold-down	Monotonic	3.5–4.9
		cyclic	3–4.2
Shear walls sheathed with GF [56]	Nailed steel brackets and hold-down	Cyclic	3.4
Shear walls sheathed with OSB and GF [56]	Nailed steel brackets and hold-down	Monotonic	5.67

Note: OSB, oriented strand board; GF, gypsum fiber.

Table 4. Ductility of modern timber walls as obtained from experimental tests.

NLTHA is still underused, likely due to the difficulties it indubitably involves and even to some inadequacies of the current codes of practice [58]. Such an analysis is, however, the better way to predict the actual seismic performance of structures composed of elastic and inelastic parts. Current codes of practice allow non-linear analyses for the calculation of the internal forces in the members of timber structures, provided that they are able to redistribute the internal forces via connections of adequate ductility [46].

When implementing a NLTHA, an efficient approach to model the structure is that of separating the critical zones where the NLTHA ductile behavior may be exhibited from the other structural parts which are expected to deform elastically even at the ultimate state. This is a typical procedure followed, for example, in reinforced concrete frames where plastic hinges are usually lumped at either end of columns and beams, while preventive plasticization of beams is guaranteed by some code-based strength hierarchy rules. An analogous procedure can be exploited for timber structures, by assuming timber members as purely elastic elements and connections as nonlinear links. To comply with the modern philosophy of the capacity design, the timber elements should be overdesigned to ensure their brittle failure will follow plasticization of connections (strength hierarchy rule).

3.2.1. Modeling timber connections

Exploiting experimental data is often the best way to obtain the mechanical behavior of a timber connection under dynamic loads. Several empirical models were proposed in the literature, which commonly involve parameters calibrated to experimental data, see for example [34, 43, 59, 60]. It should be noted, however, that extracting a general model from the experimental load-displacement curves needs caution owing to the possible dependence on both the loading history and the test set-up [34, 61, 62], as already discussed in Section 3.1.2. More detailed micro-models were also proposed by other authors, for example [62–64], which investigated the non-linear response of metallic fasteners and surrounding wood through three-dimensional finite element dynamic analyses. Still requiring some empirical adjustments of parameters, such sophisticated models usually imply a significant aggravation of the computational effort, which may become unsustainable for purposes other than those of advanced researches.

As already observed in Section 3.1, the behavior of timber connections depends from several factors, some of which are not easily predictable. This makes rather difficult to develop an analytical model able to reproduce the behavior of a timber connection. However difficult it may be, finding a suitable model for the hysteretic behavior of connections is essential to study the dynamic response of a timber structure, at least when a non-linear analysis has to be performed.

Commercial packages for structural analysis usually allow choosing between different mechanical models to implement the behavior of nonlinear links. For instance, the pivot hysteretic model provided by the widely used SAP2000 for nonlinear links (NLLINK) is depicted in **Figure 10**. To adopt a model like this, a set of parameters have to be properly assigned to reproduce all the typical phenomena experimentally detected in timber connections such as stiffness and strength degradation as well as pinching effects.

3.3. Concluding notes on the ductile behavior of modern timber structures

When appraising the ductile behavior of modern timber structures, the following aspects can be eventually addressed.

i. In-situ inspections, laboratory investigations and analytical models available in the literature converge to the statement that modern timber structures may be ductile and dissipative, provided that their metallic connections are well designed and detailed. Data collected

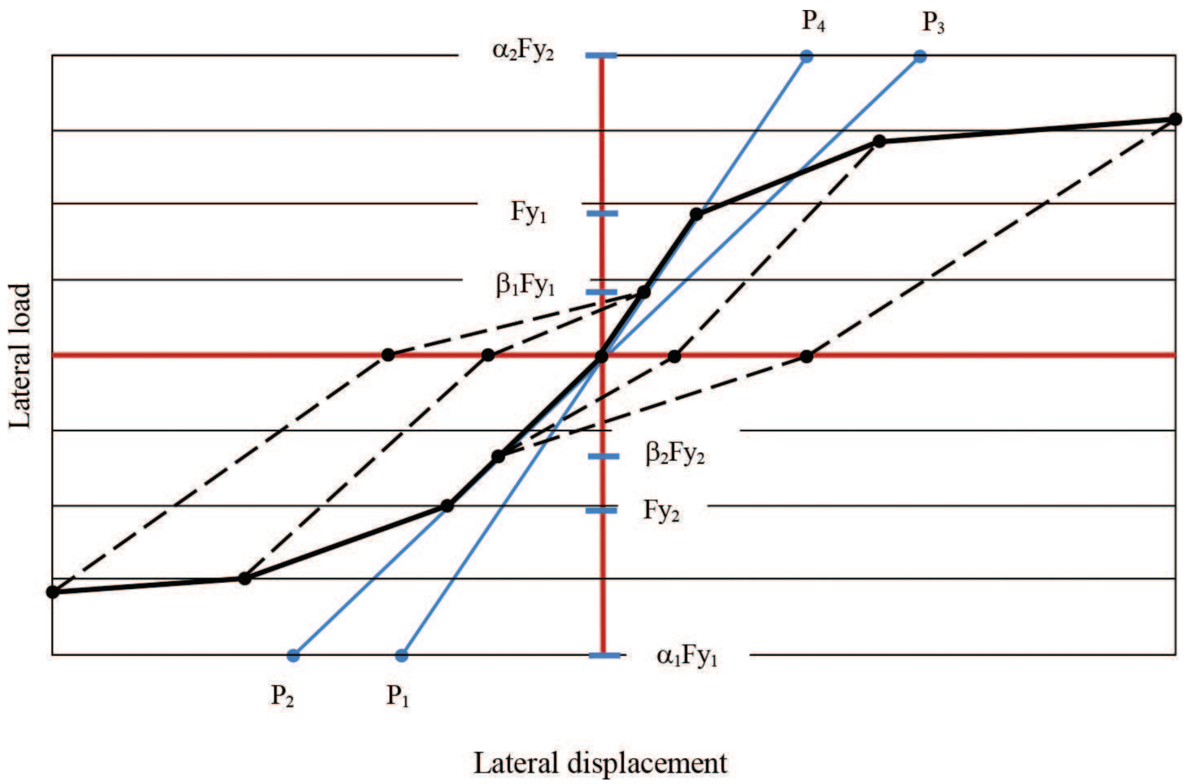

Figure 10. Multi-linear plastic pivot model for nonlinear links (NLLINK) in SAP2000.

in **Tables 3** and **4** show that timber constructions own a rather high—and sometimes excellent-ductility capacity. However, the actual ductility of a timber construction is not always univocally established. Different definitions of the ductility ratio are, in fact, proposed by authors and by codes of practice, as extensively discussed in Refs. [65–67].

ii. A decisive step to evaluate the ductility ratio, in its more classical definition, is the determination of both the deformation at yield and the ultimate deformation from the experimental load-deformation curve. Different conventional methods are available to this purpose as discussed in Refs. [66, 67], but adopting one method or another may lead to substantial differences in the calculation of the ductility ratio (even up to 100%), as evidenced in Ref. [66].

iii. The value of the ductility ratio alone cannot be sufficiently representative of the performance under strong dynamic loads. Not always, in fact, to higher values of ductility corresponds a greater amount of energy dissipated. This was addressed, for instance, in Ref. [55].

iv. To carry out a NLTHA of a timber structure nonlinear links should be located where plasticization is expected to occur, that is, at connections (plastic hinges). The mechanical model of the plastic hinges is generally difficult to be defined by the designer due to (a) insufficient support given by codes of practice; (b) difficulty in finding the experimental data relevant to the specific connection adopted; (c) uncertainty in the correct extraction of the required parameters from the experimental curves. These difficulties may eventually discourage the practitioners to carry out non-linear dynamic analyses of timber structures.

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References

[1] CEN. Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings. European Comitee for Standardization, Brussels, 2004, 232 pp.

[2] Federal Emergency Management Agency (FEMA). NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, FEMA P-750, Building Seismic Safety Commission, Federal Emergency Management Agency, Washington, D.C., 2009.

[3] American Society of Civil Engineers (ASCE). Seismic Evaluation of Existing Buildings, ASCE Standard ASCE/SEI 31-03, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA, 2003.

[4] Porcu M.C. Reducing seismic stress on buildings through inertia limiters at floor level. In: Earthquake Resistant Engineering Structures IX. WIT Transactions on the Built Environment, vol. 120, WIT Press, UK, 2013.

[5] Porcu M.C. Plastic disconnection of storey masses for seismic stress control. WIT Transactions on the Built Environment. 2015;**152**:125-133. DOI: 10.2495/ERES150101

[6] Doğangün A., Tuluk Ö.İ., Livaoğlu R., Acar R. Traditional wooden buildings and their damages during earthquakes in Turkey. Engineering Failure Analysis. 2006;**13**(6):981-996. DOI:10.1016/j.engfailanal.2005.04.011

[7] Liska A.J., Bohannan B. Performance of wood construction in disaster areas. Journal of Structural Division, ASCE. 1973;**99**(12):2345-2354.

[8] Soltis L.A. Low-rise timber buildings subjected to seismic, wind, and snow loads. Journal of Structural Engineering. 1984;**110**(4):744-753. DOI: 10.1061/(ASCE)0733-9445(1984)110:4(744)

[9] Güçhan N.S. Observations on earthquake resistance of traditional timber-framed houses in Turkey. Building and Environment. 2007;**42**(2):840-851. DOI: 10.1016/j.buildenv.2005.09.027

[10] Rainer J.H., Karacabeyli E. Performance of wood-frame construction in earthquakes. Proceedings of the 12th World Conference on Earthquake Engineering, Auckland, New Zealand. vol. 30, 2000**, p**. 1-8.

[11] Lam F., Filiatrault A., Kawai N., Nakajima S., Yamaguchi N. Performance of timber buildings under seismic load. Part 1: Experimental studies. Progress in Structural Engineering and Materials. 2004;**4**(3):276-285. DOI: 10.1002/pse.121.

[12] Vivenzio G. Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria e di Messina del MDCCLXXXIII (History and theory of earthquakes in general and in particular of those occurred in Calabria and Messina in 1733). Stamperia Regale (Royal Printing House), Naples, Italy. 1783 (in Italian).

[13] Smith W.R. Building homes to withstand hurricane damage. Forest Products Journal. 1961;**11**(6):176-177.

[14] Vivenzio G. Istoria e teoria de' tremuoti in generale ed in particolare di quelli della Calabria, e di Messina del MDCCLXXXIII (History and theory of earthquakes in general and in particular of those occurred in Calabria and Messina in 1733). Stamperia Regale (Royal Printing House), Naples, Italy. 1783 (in Italian).

[15] Hamilton G. Relazione dell'ultimo terremoto delle Calabrie e della Sicilia inviata alla Società Reale di Londra (Report on the last earthquake of Calabria and Sicily sent to the Royal Society of London). Stamperia della Rovere ("della Rovere" Printing House), Florence, Italy; 1783 (in Italian).

[16] Ruggieri N., Tampone G., Zinno R. In-plane versus out-of-plane "Behavior" of an Italian timber framed system—The Borbone constructive system: Historical analysis and experimental evaluation. International Journal of Architectural Heritage. 2015;**9**(6):696-711. DOI: 10.1080/15583058.2015.1041189

[17] Ruggieri N., Zinno R. Mechanical and constructive interpretation of the giovanni vivenzio's model. In: Ruggieri N., Tampone G., Zinno R., editors. Historical Earthquake-Resistant Timber Frames in the Mediterranean Area. Switzerland: Springer International Publishing; 2015. p. 11-20. DOI: 10.1007/978-3-319-16187-7_2

[18] Mascarenhas J. Sistemas de Construção V: O Edifício de rendimento da baixa pombalina de Lisboa. Materiais Básicos (Construction systems: The building of income of the Pombaline downtown of Lisbon. Basic materials.). Ed. Livros Horizonte, Lisbon, 2005;3 (in Portuguese).

[19] Meireles H., Bento R., Cattari S., Lagomarsino S. A hysteretic model for "frontal" walls in Pombalino buildings. Bulletin of Earthquake Engineering. 2012;**10**(5):1481-1502. DOI: 10.1007/s10518-012-9360-0

[20] Mendes N., Lourenço P.B. Seismic assessment of masonry "Gaioleiro" buildings in Lisbon, Portugal. Journal of Earthquake Engineering. 2009;**14**(1):80-101. DOI: 10.1080/13632460902977474

[21] Aktas Y.D., Akyüz U., Türer A., Erdil B., Güçhan N.S. Seismic resistance evaluation of traditional ottoman timber-frame Hımış houses: Frame loadings and material tests. Earthquake Spectra. 2014;**30**(4):1711-1732. DOI: http://dx.doi.org/10.1193/011412EQS011M

[22] Dar M.A., Ahmad S. Traditional earthquake resistant systems of Kashmir. International Journal of Civil and Structural Engineering Research. 2015;**2**(2):86-92.

[23] Ali Q., Schacher T., Ashraf M., Alam B., Naeem A., Ahmad N., Umar M. In-plane behavior of the dhajji-dewari structural system (Wooden Braced Frame with Masonry Infill). Earthquake Spectra. 2012;**28**(3):835-858. DOI: http://dx.doi.org/10.1193/1.4000051

[24] Benson T. Building the Timber Frame House: The Revival of a Forgotten Craft. New York: Simon and Schuster; 1981.

[25] Qu Z., Dutu A., Zhong J., Sun J. Seismic Damage to Masonry-Infilled Timber Houses in the 2013 M7. 0 Lushan, China. Earthquake Spectra. 2015;**31**(3):1859-1874. DOI: http://dx.doi.org/10.1193/012914EQS023T.

[26] Quinn N., D'Ayala D. In-plane experimental testing on historic quincha walls. In: 9th International Conference on Structural Analysis of Historical Constructions; F. Peña & M. Chávez (eds.), Mexico City, Mexico, 2014.

[27] Cruz H., Moura J.P., Machado J.S. The use of FRP in the strengthening of timber reinforced masonry load-bearing walls. Historical constructions – Guimarães, Portugal. 2001, p. 847-856.

[28] Santos P. Ensaios de paredes "pombalinas" (Tests on "pombalinos" walls). Technical note No. 15/97. 1997; NCE/DE, 17 LNEC Lisbon (in Portuguese).

[29] D'Ayala D., Speranza E. An integrated procedure for the assessment of seismic vulnerability of historic buildings. In: Proceedings of the 12th European Conference on Earthquake Engineering; London, UK: Elsevier Science Limited; 2003. Paper Reference 561.

[30] UNI EN 12512. 2006. Timber structures. Test methods. Cyclic testing of joints made with mechanical fasteners. UNI: Milano, Italy.

[31] Jacobsen L.S. Steady forced vibrations as influenced by damping. Transactions of ASME, 1930;**52**(15):169-181.

[32] Vasconcelos G., Poletti E., Salavessa E., Jesus A.M., Lourenço P.B., Pilaon P. In-plane shear behaviour of traditional timber walls. Engineering Structures. 2013;**56**:1028-1048. DOI: 10.1016/j.engstruct.2013.05.017

[33] Krawinkler H., Parisi F., Ibarra L., Ayoub A., Medina R. Development of a Testing Protocol for Wood frame Structures. California: CUREE Publication; 2000;W-02:1-87.

[34] Foliente G.C. Hysteresis modeling of wood joints and structural systems. Journal of Structural Engineering. 1995;**121**(6):1013-1022. DOI: 10.1061/(ASCE)0733-9445(1995)121:6(1013)

[35] Fang D.P., Iwasaki S., Yu M.H., Shen Q.P., Miyamoto Y., Hikosaka H. Ancient Chinese timber architecture. I: Experimental study. Journal of Structural Engineering. 2001;**127** (11):1348-1357. DOI: 10.1061/(ASCE)0733-9445(2001)127:11(1348)

[36] Fang D.P., Iwasaki S., Yu M.H., Shen Q.P., Miyamoto Y., Hikosaka H. Ancient Chinese timber architecture. II: Dynamic characteristics. Journal of Structural Engineering. 2001;**127**(11):1358-1364. DOI: 10.1061/(ASCE)0733-9445(2001)127:11(1358)

[37] Li X., Zhao J., Ma G., Chen W. Experimental study on the seismic performance of a double-span traditional timber frame. Engineering Structures. 2015;**48**:141-150. DOI: 10.1016/j.engstruct.2015.04.031

[38] Maeno M., Suzuki Y., Ohshita T., Kitahara A. Seismic response characteristics of traditional wooden frame by full-scale dynamic and static tests. In: 13th World Conference on Earthquake Engineering, Vancouver, Canada, 2004

[39] Suzuki Y., Maeno M. Structural mechanism of traditional wooden frames by dynamic and static tests. Structural Control and Health Monitoring. 2006;**13**(1):508-522. DOI: 10.1002/stc.153

[40] Tanahashi H., Suzuki Y. Basic concept and general formulation of restoring force characteristics of traditional wooden joints. In: Proceedings of the 13th World Conference on Timber Engineering; Auckland. 2012. p. 378-387.

[41] Makarios T., Demosthenous M. Seismic response of traditional buildings of Lefkas Island, Greece. Engineering Structures. 2006;**28**(2):264-278. DOI: http://dx.doi.org/10.1016/j.engstruct.2005.08.002

[42] Parisi M. A., Piazza M. Seismic behavior and retrofitting of joints in traditional timber roof structures. Soil Dynamics and Earthquake Engineering. 2002;**22**(9):1183-1191. DOI: 10.1016/S0267-7261(02)00146-X.

[43] Loss C., Piazza M., Zonta D. Direct displacement-based seismic design of timber structures with dowel-type fastener connections. In: World Conference on Earthquake Engineering, Lisboa, 2015.

[44] Blaß H.J., Schädle P. Ductility aspects of reinforced and non-reinforced timber joints. Engineering Structures. 2011;**33**(11):3018-3026. DOI: http://dx.doi.org/10.1016/j.engstruct.2011.02.001

[45] Quenneville J.H., Mohammad M. On the failure modes and strength of steel-wood-steel bolted timber connections loaded parallel-to-grain. Canadian Journal of Civil Engineering. 2000;**27**(4):761-773. DOI: 10.1139/l00-020

[46] EN 1995-1-2:2004. Eurocode 5: Design of timber structures-Part 1-1: General – Common 29 rules and rules for buildings. CEN, Brussels, 2004.

[47] Rodd P.D., Leijten A.J.M. High-performance dowel-type joints for timber structures. Progress in Structural Engineering and Materials. 2003;**5**(2):77-89. DOI: 10.1002/pse.144

[48] Popovski M., Prion H.G., Karacabeyli E. Seismic performance of connections in heavy timber construction. Canadian Journal of Civil Engineering. 2002;**29**(3):389-399. DOI: 10.1139/l02-020

[49] Popovski M., Karacabeyli E. Seismic performance of riveted connections in heavy timber construction. In: Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver, Canada, 2004. Paper No. 3356.

[50] Dinehart D.W., Shenton III H.W. Comparison of static and dynamic response of timber shear walls. Journal of Structural Engineering. 1998;**124**(6):686-695. DOI: 10.1061/(ASCE) 0733-9445(1998)124:6(686)

[51] Schneider J., Karacabeyli E., Popovski M., Stiemer S.F., Tesfamariam S. Damage assessment of connections used in cross-laminated timber subject to cyclic loads. Journal of Performance of Constructed Facilities. 2013;**28**(6):A4014008. DOI: 10.1061/(ASCE) CF.1943-5509.0000528

[52] Shen Y.L., Schneider J., Tesfamariam S., Stiemer S.F., Mu Z.G. Hysteresis behavior of bracket connection in cross-laminated-timber shear walls. Construction and Building Materials. 2013;**48**:980-991. DOI: http://dx.doi.org/10.1016/j.conbuildmat.2013.07.050

[53] Gatto K., Uang C.M. Effects of loading protocol on the cyclic response of wood frame shear walls. Journal of Structural Engineering. 2003;**129**(10):1384-1393. DOI: 10.1061/ (ASCE)0733-9445(2003)129:10(1384)

[54] Stehn L., Johansson H. Ductility aspects in nailed glue laminated timber connection design. Journal of Structural Engineering. 2002;**128**(3):382-389. DOI: 10.1061/(ASCE) 0733-9445(2002)128:3(382)

[55] Gavric I., Fragiacomo M., Popovski M., Ceccotti A. Behaviour of cross-laminated timber panels under cyclic loads. In: Materials and Joints in Timber Structures. Springer Netherlands, 2014. p. 689-702. DOI: 10.1007/978-94-007-7811-5_62

[56] Sartori T., Piazza M., Tomasi R., Grossi P. Characterization of the mechanical behaviour of light-frame timber shear walls through full-scale tests. In: World Conference on Timber Engineering, 2012, July, p. 180-188.

[57] Ibarra L.F., Medina R.A., Krawinkler H. Hysteretic models that incorporate strength and stiffness deterioration. Earthquake Engineering &Structural Dynamics. 2005;**34**(12):1489-1511. DOI: 10.1002/eqe.495

[58] Porcu M.C. Code inadequacies discouraging the non-linear dynamic analysis of buildings. In: Earthquake Resistant Engineering Structures XI, Alicante, Spain, WIT Press, 2017.

[59] Baber T.T., Noori M.N. Random vibration of degrading, pinching systems. Journal of Engineering Mechanics. 1985;**111**(8):1010-1026. DOI: 10.1061/(ASCE)0733-9399(1985) 111:8(1010)

[60] Dowrick, D. J. Hysteresis loops for timber structures. Bulletin of New Zealand National Society of Earthquake Engineering. 1986;**19**(20):143-152.

[61] He M., Lam F., Prion H.G. Influence of cyclic test protocols on performance of wood-based shear walls. Canadian Journal of Civil Engineering. 1998;**25**(3):539-550.

[62] Foschi, R.O. Modeling the hysteretic response of mechanical connections for wood structures. In: World Conference of Timber Engineering, Whistler, BC, Canada. 2000, July.

[63] Lo, Y.P. A comparison of two hysteretic models in predicting the response of a connector under cyclic loading (Doctoral dissertation, University of British Columbia), 2002.

[64] Lam F., Filiatrault A., Kawai N., Nakajima S., Yamaguchi N. Performance of timber buildings under seismic load. Part 2: Modelling. Progress in Structural Engineering and Materials. 2004;**6**(2):79-83. DOI: 10.1002/pse.175

[65] Jorissen A., Fragiacomo M. General notes on ductility in timber structures. Engineering Structures. 2011;**33**(11):2987-2997. DOI: http://dx.doi.org/10.1016/j.engstruct.2011.07.024

[66] Munoz, W., Salenikovich, A., Mohammad, M., & Quenneville, P. Determination of yield point and ductility of timber assemblies: in search for a harmonized approach, Proc. of Meeting 41 of CIB-W18, Canada, St Andrews, 2008.

[67] Brühl F., Kuhlmann U., Jorissen A. Consideration of plasticity within the design of timber structures due to connection ductility. Engineering Structures. 2011;**33**(11):3007-3017. DOI: http://dx.doi.org/10.1016/j.engstruct.2011.08.013