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Non-linear dynamic analysis to assess the seismic performance of cross-laminated timber structures

Maria Cristina Porcu[a,*], Claudio Bosu[b], Igor Gavrić[c]

[a] Dept. of Mechanical, Chemical & Materials Engineering, University of Cagliari, Italy
[b] MD Engineer, Cagliari, Italy
[c] CBD d.o.o. - Contemporary Building Design Company, Lopata 19g, 3000 Celje, Slovenia

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ABSTRACT

Timber structures are widely recognized to perform very well under strong ground motions. A non-linear dynamic analysis can be carried out to assess their seismic performance for design purposes, as allowed by the current regulations. When dealing with timber structures, however, the difficulties typically involved by this very powerful method of analysis may increase and sometimes become even overwhelming. The reasons are mainly due to insufficient experimental data, lack of standard constitutive models for timber connections and inadequate support provided by the current seismic codes. The key aspects related to modelling the non-linear behaviour of modern timber structures with particular focus to the crucial role of connections are analyzed in this paper. By referring to a case-study building made by solid cross-laminated (X-Lam) timber panels and designed according to the Eurocodes, the path that the designer has to undertake when carrying out a time-history non-linear analysis of a X-Lam timber structure is put in evidence, while some simplified assumptions are suggested and justified. A concentrated plasticity three-dimensional model is adopted where the hysteretic behaviour of connections is implemented by exploiting experimental data available from the literature. Three suites of earthquake records are considered in the investigation. The paper aims to give structural engineers some useful hints to carry out non-linear dynamic analyses of cross-laminated timber structures for design purposes.

1. Introduction

The capacity of deforming without collapsing under horizontal actions is the main requirement that structures built in seismic areas should possess, no matter if they are reinforced concrete, steel, masonry or timber buildings. To meet such a requirement, modern structures are asked to be ductile, i.e. to be able to undergo large deformations (exceeding the elastic limit) without significant reduction in strength. A ductile behaviour, in fact, offers many benefits among which are the great amount of input energy that can be dissipated during the inelastic motion and the beneficial stress redistribution that can be achieved in structural systems. Seismic codes typically require that structures possess inelastic deformation capacity and to this purpose, they provide detailed rules to guarantee sufficient local and global ductility, as well as the fulfillment of the hierarchy of strengths [1].

To assess the ductile and dissipative behaviour of structures under strong earthquakes, non-linear methods of analysis can be adopted by the structural designer, in accordance with the current regulations. Among them, the non-linear time-history analysis (NLTHA) [2,3] is the most powerful, based as it is on the direct numerical integration of the motion equations under acceleration histories consistent with the code-based elastic response spectrum. The considerable effort required to the structural engineer, however, makes the NLTHA method still scarcely adopted in the design practice. Lacks or inadequacies of the seismic codes may add further reasons to the distrust of this very effective method of analysis, as discussed in [4].

The present paper shows that when a timber structure is taken into account, the difficulties that the designer has to face in order to perform a NLTHA may become even greater. Experimental data are essential to model the inelastic behaviour of timber structures, due to the key role played by connections, the hysteretic behaviour of which is not easily predictable by means of simple analytical models [5]. Although wood is an inherently-brittle material, wood-made structures can exhibit significant ductility, provided that well-designed metallic connections join the timber members [5–7]. Several empirical or semi-empirical models were proposed in the literature to reproduce the behaviour of metallic connections (accounting for stiffness and strength degradation as well as pinching effects), that commonly involve parameters calibrated to

* Corresponding author.
E-mail address: mcporcu@unica.it (M.C. Porcu).

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experimental data [5,8–11]. However, extract a general model from the experimental load-displacement curves requires caution owing to the possible dependence of the hysteretic cycles on both the loading history and the test set-up, [5,12,13]. On the other hand, experimental studies on timber connections are still limited and the current codes of practice generally do not provide standard constitutive models. The reader may refer to [14,15] for an extensive discussion on the state-of-the-art of the modelling approaches accounting for the hysteretic behaviour of the metallic timber connections.

Based both on experimental evidences and analytical results available from the literature, the present paper aims to provide the practicing engineers a feasible approach to carry out code-based non-linear dynamic analyses of X-Lam structures. The software package SAP2000 [16] is used to this purpose, since it has been shown to be quite effective in the non-linear analysis of X-Lam structures, see e.g. [17,18]. By considering a two-storey building made by X-lam panels and by referring to the Eurocode 8 (EC8) provisions [2], the paper covers all the steps that the designer should follow to carry out a NLTHA of a X-lam building. Typical features of the behaviour of X-Lam panels and of their metallic connections will be discussed while simplifying assumptions will be suggested and justified. To explore the extent of the ductile capacity of timber buildings, suites of earthquake records with different peak ground accelerations are considered in the investigation.

2. Case-study building

An existing residential two-storey building made by X-Lam solid wood panels and built by STP srl company [19] in the area of Modena, Italy, is considered in the study, see Fig. 1. It was designed through the modal response spectrum analysis according to EC8 [2] and Eurocode 5 (EC5) [20]. The building walls are made of X-Lam panels type CLT_100_C3S_NVI_C24, consisting of three spruce wood C24-class layers oriented crosswise to each other, see Table 1 for more details. The floor slab is made by X-Lam panels type CLT_160_L5S_NVI_C24, composed of five spruce wood C24-class layers oriented crosswise to each other and arranged symmetrically to the middle layer (see Table 1). A 30 cm thick reinforced concrete basement floor slab acts as foundation of the building, while the X-Lam walls are placed on 20 cm high concrete foots. The roof is made by 16 × 30 cm glulam GL24h principal rafters and 12 × 16 cm solid spruce wood C24-class joists. Table 2 lists the metallic connections adopted in the building. Fig. 2 shows the location of the angle-brackets and hold downs in the building plans. The roof plan and a vertical section of the building are also provided in Fig. 2.

3. Concentrated plasticity 3D model

3.1. Main assumptions

The finite element program SAP2000 [16] was used to model the building (see Fig. 3a) and carry out the non-linear time-history analyses. A concentrated plasticity three-dimensional model was adopted, in analogy to what is commonly done when modelling the post-elastic behaviour of reinforced concrete or steel frames. Concentrated plasticity models are, in fact, usually adopted to model X-Lam buildings, see for instance [14,24,25]. Based on the generally accepted evidence that the ductile capacity of X-Lam buildings mainly resides in the metallic connections [14,22,23], this modelling approach assumes that the critical sections where the ductile behaviour is expected to occur are the metallic connections (hold-downs, angle-brackets, screws and bolts) between adjacent wall-to-wall, wall-to-floor or floor-to-floor panels and between timber panels and foundations.

Table 1
Characteristics of X-Lam panels.

	Panel type	Thickness (mm)	Layers' thickness (mm)	Strength class [21]
walls	CLT_100_C3S_NVI_C24	100	30–40–30	C24
floor	CLT_160_L5S_NVI_C24	160	40–20–40–20–40	C24

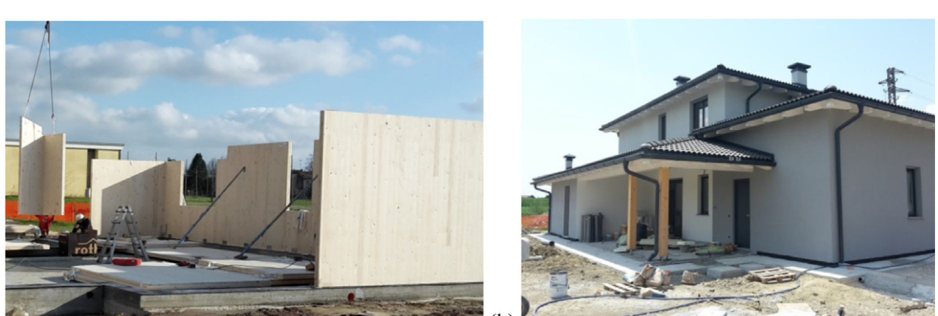
A lumped mass matrix is assumed by SAP2000, entailing all the masses associated to the degrees of freedom (DOF) of the model's nodes. According to Section 3.2.4 of EC8 [2], the following combination of actions was considered to evaluate the inertial effects:

$$G_1 + G_2 + \psi_{2i} Q_{ki}. \tag{1}$$

Here G_1 and G_2 are structural and non-structural permanent loads, respectively; Q_{ki} denotes the variable actions and ψ_{2i} is a coefficient taking into account the likelihood of variable actions. The masses associated to the weight of structural elements are calculated by default by SAP2000, while the masses relating to non-structural elements and variable loads should be calculated externally and added to the analysis. The values assumed herein are provided in Table 3.

Under the hypothesis of Rayleigh's classic damping, the damping matrix was obtained as a linear combination between the mass matrix and the stiffness matrix by assuming a constant damping factor $\xi_i = \xi = 0.05$ (5%) for all modes. In timber buildings, the value of the viscous damping ratio ranges typically from 2% to 5% [24,26]. The highest value of this range (5%) was chosen in the present investigation to implicitly (and roughly) take into account the energy dissipation due to friction at joints, not explicitly introduced in the model (see Sections 3.4 and 3.5). Since higher values of damping ratio may result in underestimating some response quantities, while accurately predicting others, cf. [26], the designer should be aware that the assumed value of the elastic-viscous damping ratio can influence the numerical analysis results. It can be also considered, on the other hand, that the more inelastic the response becomes (which typically happens under severe earthquakes) the less it is influenced by the elastic part of the motion, cf. [27].

The Hilber-Hughes-Taylor method [28] is adopted by default by SAP2000 to integrate the motion equations. It defines parameters β and γ as a function of a single parameter α:



(a) (b)

Fig. 1. The residential case-study building (a) under construction and (b) completed.