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Elastic and ductile design of multi-storey crosslam massive wooden buildings under seismic actions

M. Fragiacomo [a,*], B. Dujic [b], I. Sustersic [b]

[a] Department of Architecture, Design and Urban Planning, University of Sassari, Palazzo del Pou Salit, Piazza Duomo 6, 07041 Alghero, Italy
[b] CBD d.o.o. - Contemporary Building Design Company, Lopata 19g, 3000 Celje, Slovenia

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ABSTRACT

The paper discusses the seismic design of multi-storey buildings made from cross-laminated timber panels (‘crosslam’). The use of seismic analysis methods such as the modal response spectrum and the non-linear static (push-over) analysis is discussed at length, including issues such as the modelling of crosslam walls and connections, the evaluation of the connection stiffness, and the schematization of floor panels. It was found that it is crucial to account for the flexibility of the connections (hold-downs and angle brackets) between upper and lower walls, since otherwise the vibration periods of the building would be underestimated. The basics of capacity design to ensure the attainment of ductile mechanisms in crosslam timber structures under seismic actions are presented. The ductile failure mechanism is characterized by plasticization of connectors (hold-downs, angle brackets and screws) between adjacent wall panels and between panels and foundations. The crosslam panels and the connections between adjacent floor panels must be designed for the overstrength of the connectors to ensure that they remain elastic during the earthquake and the ductile failure mechanism is attained. Based on the results of preliminary quasi-static cyclic tests, a value of 1.3 was found for the overstrength factors of hold-downs and angle brackets. A case study multi-storey crosslam massive wooden building was then analysed using the non-linear push-over analysis as implemented in the N2 method recommended by the Eurocode 8. The building was modelled using shell elements and non-linear links to schematize the hold-downs and angle brackets. The building ductility, calculated from the bilinear curve equivalent to the actual non-linear push-over curve, was then investigated. Such a quantity, defined as the ratio of the displacement at the near collapse state and the maximum elastic displacement of the top floor, was found to rise from 1.7 to 2.5 when ductile instead of brittle hold-downs and angle brackets are used. Furthermore, the maximum peak ground acceleration the building can resist raised from 0.2*g* to 0.4*g*, demonstrating the importance of using ductile connectors in seismic design.

1. Introduction

Different methods can be used for the design of seismic-resistant buildings. The most basic approach would be to evaluate the forces induced on a building by an earthquake with a high return period and design the structure in elastic phase. Since statistically the chance of a high intensity earthquake occurring during the lifetime of a building (in most cases 50 years) is not particularly high (about 10%), the elastic design leads to significant overdesign of the building elements. For this reason the elastic approach is generally used only in low to moderate seismicity regions. The alternative design approach is based on the principles of ductile design. A ductile structure is able to dissipate energy during the seismic event by undergoing through plastic deformation. One of the advantages is the possibility to survive high intensity earthquakes as long as the displacement demand in the ductile parts of the structure does not exceed the displacement capacity. The ductility also allows more economical structures to be built as the design seismic actions can be reduced depending upon the ductility ratio [1]. Such an approach is generally followed for building design in medium to high seismicity regions.

Current codes of practice [2] suggest two different approaches for design of ductile structures in earthquake-prone regions. The first approach, well known and widely used, is referred to as the Force-Based Design (FBD) method since it mainly focuses on designing the strength of the structure [1]. The objective is the evaluation of the behaviour factor q, which is employed to transform the elastic response spectrum into a design spectrum. In this way a non-linear structure can be designed using a linear-elastic static or dynamic (modal response spectrum) analysis under seismic action, with the structural ductility only implicitly considered when evaluating the behaviour factor q.

* Corresponding author. Tel.: +39 079 9720418; fax: +39 079 9720420.
E-mail addresses: fragiacomo@uniss.it (M. Fragiacomo), bruno.dujic@cbd.si (B. Dujic), iztok.sustersic@cbd.si (I. Sustersic).

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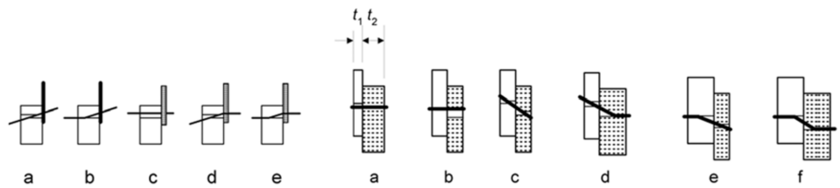

Fig. 1. Failure modes for steel–timber (left) and timber–timber (right) connections, notations according to EC5-1-1 [15].

The second approach, which explicitly refers to the structural ductility in addition to the strength, is based on a Non-linear Static Analysis (NSA) procedure [1]. The purpose of this approach is the evaluation of the actual structural response mainly in terms of ductility demand and, hence, ultimate displacement induced in the structure by the earthquake ground motion [3]. A number of different methods have been proposed, including a modified version of the N2 method [4], which has been adopted by the new Italian regulation [5] and by the Eurocode 8 (EC8) [2] and is discussed more in detail in the following sections.

An important issue in seismic design is the identification of suitable, ductile failure mechanisms. Capacity-based design must then be used [6] to ensure that brittle failure mechanisms will not occur. For multi-storey timber buildings with lightframe construction, the most ductile failure mechanism is shear in the nailed/screwed connections between the sheathing and the frame [7]. All other failure mechanisms are fairly brittle and, therefore, should be avoided by designing the corresponding members (hold-downs, bolts, timber studs, timber plates, plywood sheathing) for the overstrength of the nailed/screwed connection.

New structural systems for multi-storey timber buildings have recently been proposed in Europe and in Australasia. Unlike lightframe construction, these innovative systems can also comply with the new philosophy of the Damage Avoidance Design, according to which a building should not only survive an earthquake at ultimate limit state, but be easily repairable and useable in a short time so as to reduce the disruption and the associated cost to a minimum [8]. In Australasia, hybrid systems made of laminated veneer lumber (LVL) walls and frames prestressed with unbonded tendons used together with energy dissipaters were developed [9,10]. In such systems the prestressed tendons keep the timber elements connected together and ensure that after an earthquake event, due to the restoring force, the structure returns to its initial position with little, if not, residual deformation. The energy dissipaters ensure proper dissipation with reduction in displacement demand to the structure. In Japan, multi-storey buildings constructed from prefabricated walls and slabs made of cross-laminated timber ('crosslam') produced in Europe were subjected to full-scale shaking table tests. The buildings survived high intensity earthquakes with limited structural damage [11,12]. Furthermore, it was found that such systems, with a proper choice of connection details and panel sizes, can dissipate a significant amount of energy, mostly in the connections between wall panels, and between panels and foundation, leading to the possibility of carrying out static and modal response spectrum analysis assuming a behaviour factor $q = 3$ [13].

Despite the extensive use of the crosslam technology, there are few provisions for the seismic design of this system in current codes of practice such as the Eurocode 8 [2]. In addition to the lack of any value for the overstrength factor, there is no suggestion on the choice of the ductile failure mechanism, nor indication on the way the NSA can be carried out according for example to the N2 method. This paper provides some answers to the aforementioned queries, namely it proposes some values of the overstrength factors for typical connections used in crosslam construction based on the results of experimental cyclic tests, and presents the use of the NSA in the design of a simple crosslam building. Further important issues such as the numerical modelling of crosslam panels and connections, the influence of the connection ductility on the global ductility of the building, and the seismic performance of the whole building are critically discussed in the paper.

2. Choice of an appropriate ductile failure mechanism for multi-storey crosslam buildings

2.1. Provisions for a ductile connection between crosslam panels

Crosslam panels are solid slabs made from layers of timber boards with the adjacent layers glued at a right angle. Advantages over glued-laminated elements include improved stability in both directions, which is of particular importance for 2D elements, and the possibility to use medium to low quality timber. This technique was developed in Europe about 15 years ago and is nowadays extensively used. Crosslam buildings are erected by connecting crosslam walls and slabs together using angular metal bracket connectors, usually nailed or screwed to the timber, and self-tapping screws. The connections between adjacent panels are usually realized using 8 mm diameter screws placed at a distance of 300 mm centre to centre [14], where the penetration of the screw in the second wall is usually equal or longer than the penetration in the first wall. The foundation–panel and upper–lower panel connections are made from nailed or screwed angular metal brackets, and in some cases also by hold-down connectors, nailed and bolted to the timber and reinforced concrete foundation at the first storey. The wall-to-floor panel connections are often made of 8 mm diameter screws placed at a distance of 300 mm centre to centre.

Different failure mechanisms can occur in a crosslam building, and only few of them are ductile. Failure of the wall panel due to in-plane loading (shear, bending and axial force) is mostly brittle and should be avoided by designing the panel for the overstrength of the ductile elements (the connectors). The connectors between adjacent panels and between panels and foundation, however, may behave ductile or brittle under shear deformation depending on whether plasticization of the steel fastener (screws and nails) is attained or not. Therefore, according to the notation of the Eurocode 5 Part 1-1 [15], see Fig. 1(left), failure mode 'b' is regarded as ductile (one plastic hinge formation), whilst mode 'a' that occurs with shorter and/or thicker fasteners and has no plastic hinge formation in the steel fastener is regarded as brittle. The most desirable failure mechanism would be mechanism 'e', where two plastic hinges are formed in the fastener. However, this would only be possible if thicker steel plates were used, which does not apply to typical brackets and hold-downs. For screwed connections between adjacent panels, mode 'f' is the most ductile and therefore the most desirable. Modes 'd' and 'e' are also ductile though only one plastic hinge is formed (Fig. 1(right)).

2.2. Definition of the overstrength factor

An important issue in seismic design is the identification of suitable, ductile failure mechanisms. Capacity-based design must then be used [6] to ensure that brittle failure mechanisms will