Seismic Performance of Cross-Laminated Timber Panel Buildings with Dissipative Connections

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SUMMARY:
Cross-laminated timber panel buildings are gaining a growing interest of the scientific community due to significant technical advantages, such as the material sustainability, the high fire resistance and quickness of erection. Nevertheless, it is well known that timber panels themselves are not able to dissipate a significant amount of energy during an earthquake. In fact, in this system the seismic design is carried out in order to dissipate the energy by means of inelasticity of connections. Generally, the elements devoted to withstand plastic deformations are the panel-panel and panel-foundation joints and, therefore, their ability to sustain repeated excursion in plastic range governs the building inelastic response. The paper here presented aims to propose an advanced approach for designing cross laminated timber panel buildings. In particular, it is proposed to substitute the classical hold-downs, which usually exhibit a limited dissipation capacity, with an innovative type of dissipative angle bracket. The new connections, called dissipative L-stub, apply the concept usually adopted for designing the hysteretic metallic dampers ADAS (Added Damping and Stiffness). In particular, their tapered shape allows a better spread of plasticization resulting in a high dissipation capacity. Within this framework, in order to characterize the force-displacement response under cyclic loads of L-stubs an experimental campaign is carried out. Afterwards, the effectiveness of the proposed approach is proved by analysing the non-linear response under seismic loads of a three-storey building alternatively equipped with hold-downs or L-stub. Finally, the response of classical and innovative system is compared in terms of behaviour factor.

Keywords: timber structures, Cross-laminated Panels, cyclic tests, non-linear dynamic analyses, q-factor

1. INTRODUCTION

Timber is increasingly being used as structural material also in Europe. The sustainability and the technical advantages of modern wooden buildings are related to the fact that they are quick to erect, with high fire resistance and give the possibility to be integrated in energy efficient solutions. All these features make timber buildings very appropriate not only for new buildings but also for emergency cases such as the post-earthquake reconstruction, as happened in L'Aquila in 2009 (C.A.S.E. project). In addition to these advantages, it is demonstrated that timber structures can exhibit good seismic performance deriving from a correct design of wood members and connections. From the structural standpoint, timber buildings assembled with steel joints combine the advantages of the two materials. In fact, the high resistance/weight ratio of wood reduces the horizontal seismic actions, while the steel parts (fasteners, nail, screws, etc.), which can exhibit a ductile behavior, are employed to dissipate the input energy through their inelastic response.

Among the various timber structural systems, cross-laminated timber panel buildings, despite their recent introduction, have rapidly spread on the Italian and European market, which for a long time has been oriented on structural solutions adopting masonry or concrete rather than lightweight material such as steel or timber, that are mainly used for transitional buildings or roof covers. From the seismic standpoint, a timber panel building is conceptually similar to a box structure in which walls and floors are rigid in their plane. Usually the structure is constituted by the assemblage of precast flat cross laminated panels used for realizing both the vertical resistant system and slabs. The vertical panels are connected to each other by means of screws, nails and angle brackets. As far as the panels behave as rigid elements the deformation of the whole structure is usually lumped in the connections. Generally each panel at the ends possesses at least a couple of long nailed angles, namely Hold-downs, devoted to prevent the rocking and a number of smaller angles devoted to absorb shear forces.

Despite the advantages aforementioned, there is still a lack of knowledge of the true structural behavior of the system and more efforts of the scientific community are still needed. In fact, up to now the European codes do not contain specific guidance on fabrication, design and procedures of inspection and maintenance, although these have already been developed in practice. Furthermore, there are not satisfactory rules for steel joints subjected to seismic loads and the behavioral factor (q-factor) proposed by the code are not specifically devoted to timber panel buildings. Recently a great experimental effort has been dedicated to this structural system in Italy. In fact, in (Ceccotti et al.,2000) several tests on typical joints, on structural sub-assemblages and on a three-storey building have been carried out. Basing on the obtained results, Ceccotti et al. (2006) have carried out a set of numerical time-history analysis on the tested building aiming to evaluate the behavioral factor. On the base of these analyses, the value of the q-factor proposed by the authors for X-lam buildings employing classical fastening details is equal to three. Even though the value found in (Ceccotti et al.,2006) is referred to a single case study, it demonstrates a low-dissipative capacity of timber panel buildings compared to other structural systems. It is easy to understand that, since the elements mainly devoted to the energy dissipation are the Hold-downs, the reasons of such an unsatisfactory behavior can be searched in their response under cyclic loads. In fact, as already demonstrated in technical literature, Hold-downs subjected to cyclic loads exhibit a response characterized by significant pinching resulting in a low capacity to dissipate energy.

Within this framework, in this paper, the authors present a new type of dissipative connection, called in the following "XL-Stub", which has already been successfully applied to steel structures, and now is adapted to timber panel buildings in substitution of the typical hold-downs. In particular, in the following paragraphs, the experimental monotonic and cyclic tests carried out at the University of Salerno on the XL-Stub are compared with the results of cyclic tests on hold-downs with same resistance. Test results are then used to carry out a numerical analysis on the three-storey buildings tested within the SOFIE project. In particular, the aim of the work is to compare the performance of the building alternatively equipped with hold-downs or XL-Stubs by means of Time-History analysis. Therefore, the structural model carried out by means of the software "Seismo struct" (Seismosoft, 2011)) is first calibrated on the experimental test on the whole building and is then used to perform Incremental Dynamic Analyses.

2. CYCLIC BEHAVIOUR OF TYPICAL CONNECTIONS

Figure 1. Hold-down, angle bracket and screwed-joints

Timber panel buildings are usually constituted by the assembly of panels made of cross-laminated wood, connected together through specific fastening elements. The classical systems of connections are:

- *Hold-downs*, which are subjected mainly to tension loads and prevent the uplift of walls from foundations or floors;
- *Angular brackets*, which are mainly subjected to shear loads and prevent the sliding of the walls;
- Screwed-joints, which prevent the relative movement between the panels of the walls.

As mentioned before, in case of timber panel buildings, the behavior under cyclic loads of connections strongly influences the overall structural response. In particular, ductility and energy dissipation capacity of joints are mainly governed by the dissipative characteristics of the element which first undergoes plastic deformation. In case of angles, such an element may be the fastener (nail, screw, etc.) or the angle itself. Typical behavior of joints employed in timber panel buildings is characterized by a strong plate-weak connector behavior and, therefore, the dissipative response is governed by the localized crushing of the wood and by the plasticization of fasteners which fails in bearing.

In the following, reference is made to the connections employed in the building tested within the SOFIE project. All three types of joints have been already studied by means of tensile cyclic tests by Gavric et al. (2011). In this work, the Hold-down WHT540 with 12 annular ringed nails 4x60 mm, whose geometry and thickness is very similar to that of the Hold-down HTT22 originally used in SOFIE project, has been tested. Furthermore, the BMF 90x116x48x3 mm angle bracket with 11 annular ringed nails 4x60 mm and two different types of panel-panel screwed connections have been tested.

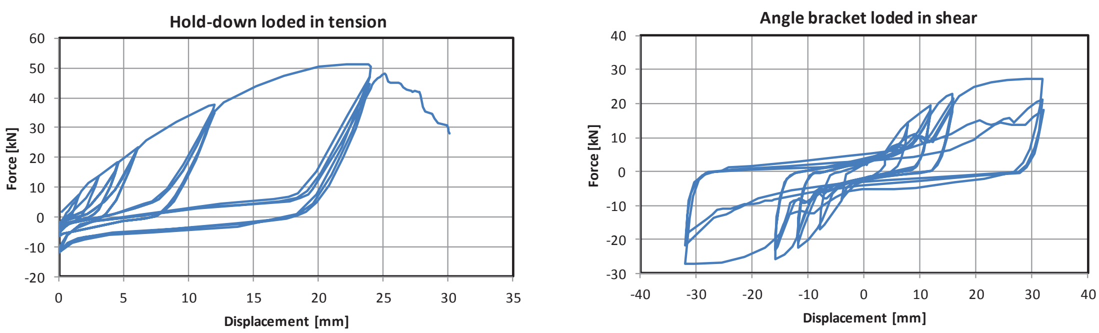


Figure 2. Hysteresis loops of a hold-down loaded in tension and of an angle bracket loaded in shear

Cyclic tests demonstrate that all connections are characterized by significant pinching phenomena (Fig.2). In fact, both Hold-down and angle bracket show a low energy dissipation capacity, significant stiffness and strength degradation. Nevertheless both joints appear to exhibit a significant ductility (24.5 mm for the hold-down and 30 mm for the angle). Pinching behavior is mainly due to the mechanism of plasticization. In fact, during the loading history, the holes ovalize due to the bearing forces and, therefore, at the load reversal the nail has to slide into the hole before going in force.

3. CYCLIC AND MONOTONIC TESTS ON XL-STUBS

The ADAS devices (Added Stiffness And Damping) are commonly used in passive control techniques in order to absorb seismic input energy by means of their application, for example, on V-bracing. What makes these devices, but more in general all hysteretic metal dampers, particularly suitable for this type of application is their shape. In fact, ADAS work in double curvature and are designed aiming to exploit the dissipative characteristics of the steel along all the extension of the device, giving a proper shape to the metal plate.

XL-stubs are based on the same principle, presented here and tested with monotonic and cyclic loading tests with constant amplitude (15, 25 and 30 mm) and variable amplitude at the University of Salerno. They are the extension of the concept of T-stub with double curvature suggested by Latour & Rizzano (2012) to angle steel sections. In fact, the flanges of the angles are carved in the area between the web and the bolts through oxyfuel, as a dissipative T-stub. The design of the XL-stub was carried out in order to have a resistance which is equal to that of the hold-down tested in Gavric et al. (2011). Therefore, five pairs of XL-stubs were connected with eight M12 bolts of 8.8 class to a panel of larch, which belongs to the family of conifers such as spruce, with which are made commonly timber panel buildings. The connection has been designed according to the Eurocode 5, assuming the dimensions of the panel in such a way that the thickness is $b_e \geq 6d$, the width deriving from the spread at 30° of stress from the outermost row of bolts and the height was the maximum compatible with the testing machine (Schenck Hydropuls S56, maximum load 630 kN, maximum displacement of + / - 125 mm).

The five pairs of XL-stubs were subjected to a tension monotonic test (A05-M), three tension cyclic tests with constant amplitude of 15, 25 and 30 mm (A01-C15, C25-A02, A03-C30) and a variable amplitude cyclic test (CV-A04). The L-stubs were connected to the wooden panel as mentioned above, and were fixed down to the rigid support with four M18 bolts 8.8 class (2 for L-stub), while were fixed up to the other rigid support with two steel plates welded to this.

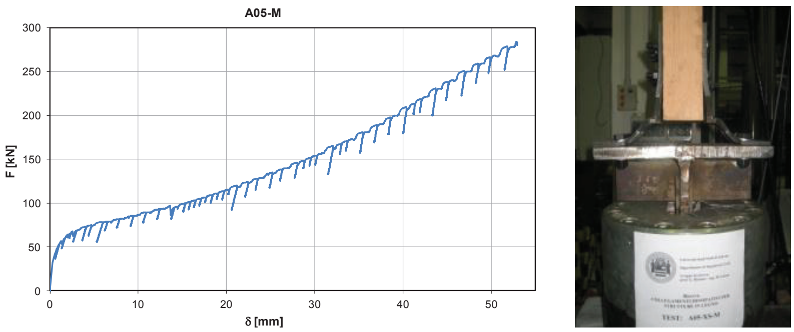


Figure 3. Monotonic curve and test setup for tension tenting of L-stubs connections

The tests were conducted under displacement control, with a variable speed from 0.3 mm/s to 0.6 mm/s for the cyclic tests with constant amplitude (0.01 Hz), from 0.5 mm/s to 0.9 mm/s for the variable cyclic test (according to AISC 2005 for cyclic loading tests of connections) and with a constant speed of 0,025 mm/s for the monotonic test.

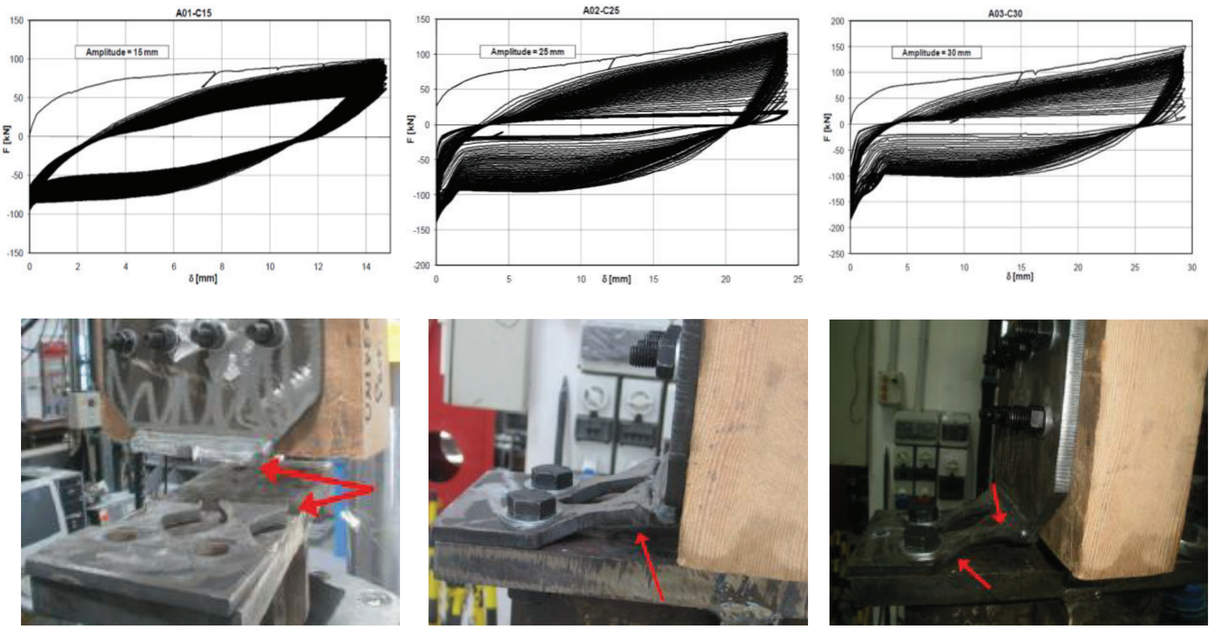


Figure 4. Hysteresis loops and failure mode of a couple of XL-stubs in constant amplitude cyclic tests

The collapse of the links in the three tests with constant amplitude was reached at the weld between the web bolted to the panel and the dissipative flange. In particular, in the test with constant amplitude of 30 mm the failure happened almost simultaneously in the two opposite sections of the neck-in. The cracking affected progressively the width and the thickness of the flange from the welding, with loss of strength, stiffness and dissipation capacity before the collapse. The measurement of heat on the flange of L-stub in the first six cycles of the test with amplitude of 25 mm, confirms the mechanism of complete plasticization of the steel of the flange.

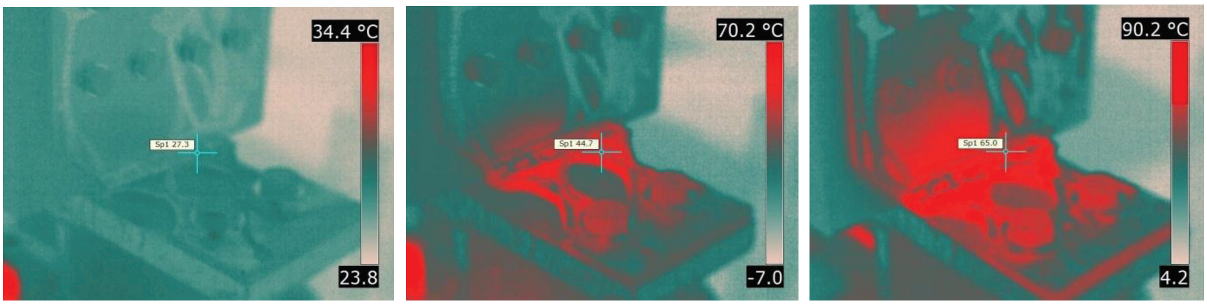


Figure 5. IR camera pictures of the L-stub up to the sixth cycle at 25 mm of amplitude

From the results obtained with the cyclic and monotonic tests on the L-stubs, the Wöhler low-cycle fatigue curve of the link was built. The condition of conventional collapse was fixed in correspondence of the cycle which corresponds to a reduction of 50% of the capacity to dissipate energy (Castiglioni and Calado, 1996). As shown in Figure 6, the number of cycles for the conventional failure of the specimens in the three cyclic tests with constant amplitude of 15, 25, and 30 mm is respectively 107, 43 and 29.

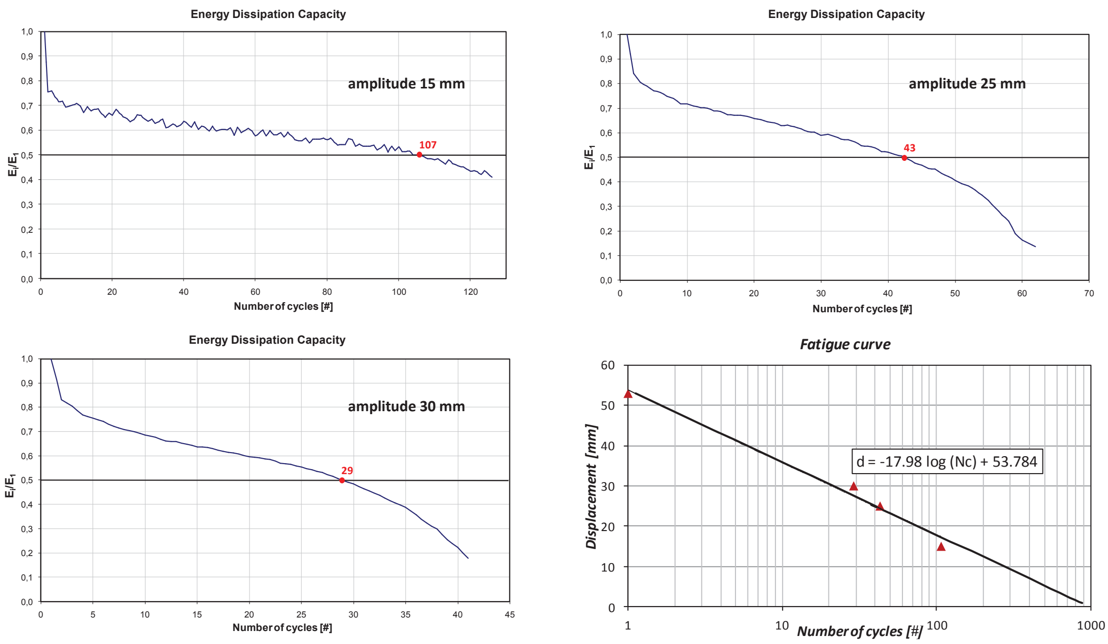
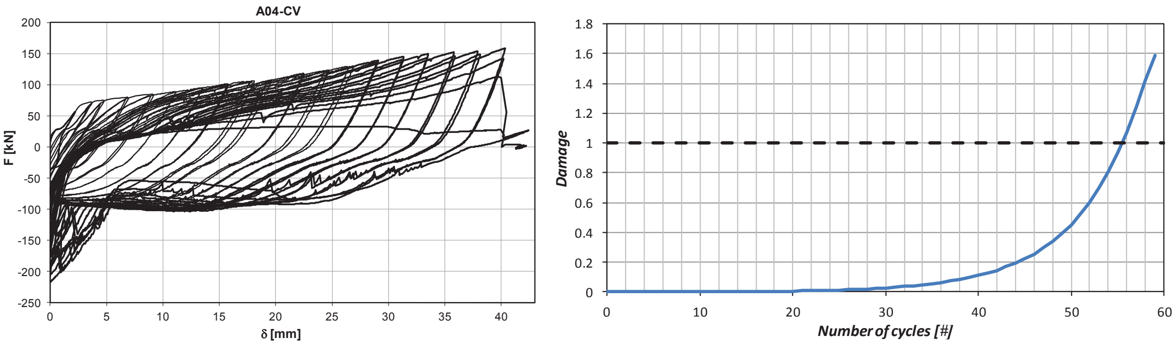


Figure 6. Conventional collapse of specimens under constant amplitude cyclic tests and fatigue curve

These points, together with the ultimate displacement reached in the monotonic test, allow to build the low-cycle fatigue curve of the joint. The prediction of the damage *D* for variable amplitude cyclic test (Fig. 7) was carried out by applying Miner's rule:

$$D = \sum_{i=1} \frac{n_i}{n_c} \tag{3.1}$$

where n_i is the number of cycles carried out at for a given amplitude and n_c is the number of cycles for which the joint reaches the failure ($D = 1$) at the same amplitude, deduced from the fatigue curve. In Fig.7b the curve of the cumulative damage is clear that the conventional failure of the specimen for the variable amplitude cyclic test has occurred at 56-th cycle, while it performed 58 complete cycles before it has showed a sudden reduction of the load bearing capacity and the complete fracture of the web of the L-stubs.



(a) (b)

Figure 7. Hysteresis loops of specimen under variable cyclic test and cumulative damage

The comparison with the cyclic behavior of the hold-downs normally used in cross-laminated timber panels buildings shows clearly as the L-stubs have much better performance in terms of dissipation of energy.

4. INFLUENCE OF CONNECTIONS ON SEISMIC RESPONSE OF TIMBER PANEL BUILDINGS

The q-factor which is used in the design to reduce the forces obtained from linear analyses in order to take into account the non-linear dissipative behavior of the structure, depends on the material, the structural system and the design method adopted. There are several procedures to determine the q-factor for a given structural system and any method requires:

- a campaign of cyclic tests on typical structural elements of the building system aiming to derive the information necessary for the calibration of proper numerical models;
- an appropriate three-dimensional numerical model able to simulate the non-linear response in the time domain of a structure submitted to a proper set of real seismic events;
- the availability of the results of shaking table tests on a real building in order to calibrate the numerical model;
- the definition of an appropiate yield criteria and collapse criteria of the structure.

In this work, starting from cyclic tests on the connections available in literature and from studies carried out on L-stubs at the University of Salerno, a numerical model of a full scale three-story building tested on shaking table was created, and a number of non-linear dynamic analysis (IDA) by means of the finite element code SeismoStruct of the software house SeismoSoft are implemented, subjecting the model to six natural accelerograms. The q-factor was then calculated as the ratio:

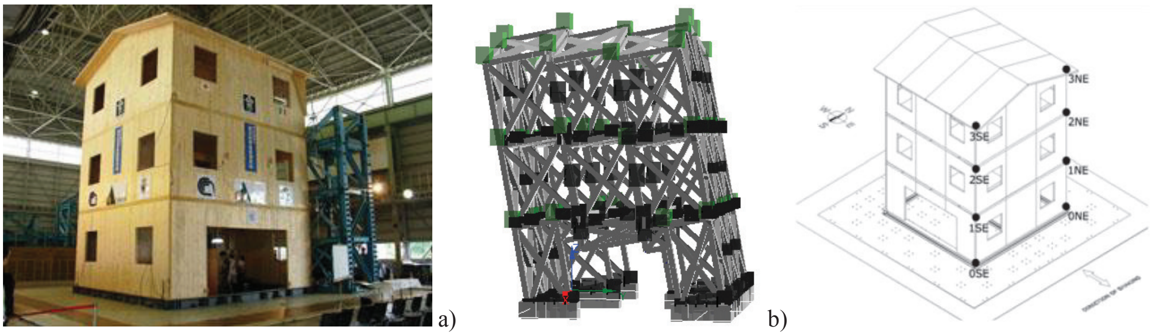
$$q = \frac{\alpha_u}{\alpha_y} \tag{4.1}$$

where α_u is the acceleration multiplier which leads to structural collapse and α_y is the acceleration multiplier corresponding to the threshold of the first yield of the building, so as to bring into account in both the multiplier the frequency content of the earthquake in relation to the characteristics of the building.

4.1. The numerical model of the building

The building analyzed is the same tested in full scale on shaking table in July 2006 in the laboratory of the National Institute for Earth Science and Disaster Prevention (NIED) in Tsukuba, Japan, as part of SOFIE research project (Construction System Fiemme), sponsored and conducted by the Trees and Timber Institute (IVALSA) of the National Research Council (CNR) and funded by the Province of Trento, Italy (Fig.8). The three-story building has plan dimensions of about 7×7 m, is 10 m height and has a gable roof. The walls are 85 mm thickness and are made with three cross-laminated timber panels (XLam) connected with screwed-joints with LVL strips. The hold-downs, replaced by L-stubs in the simulations carried out, are arranged at the ends of the walls and near the openings, to connect the panels with the foundation and with the floors. Two angle brackets for each panel also connect the walls with the foundation and with the floors. The two floors are made by 142 mm thickness panels. On the two slabs were placed additional masses in order to simulate the weight of the additional dead loads and the 30% of variable loads. On the roof, instead, there are not additional loads according to the Eurocode 8 and to the Italian NTC2008.

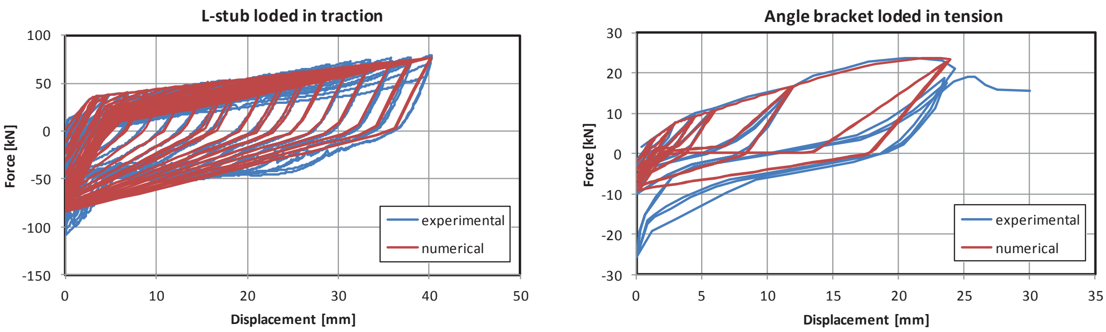
The simulation has been performed on the building characterized by an opening of 2.55×2.55 m on the ground floor. Non-linear dynamic analyses of this configuration with hold-downs are already available in literature (Ceccotti et al., 2006). The numerical model was implemented in the software SeismoStruct, assuming a rigid behavior of the panels and of floors in their plane. So each panel was modeled as an equivalent rectangular undeformable frame.



a) b) c)

Figure 8. a) Real building; b) deformed configuration of the numerical model; c) points of control of displacements

Starting from the results of cyclic tests on Hold-Downs connections and on XL-stubs, multilinear cyclic models were calibrated in order to reproduce the cyclic behavior of the connections loaded in tension and in shear. This analytical model allows to simulate the stiffness and strength degradation, and the pinching phenomenon of the joint hysteretic curve, through the definition of the 15 parameters reported in Table 4.1. In Figure 9, the numerical modesl are compared with the experimental curves of the connecting elements.



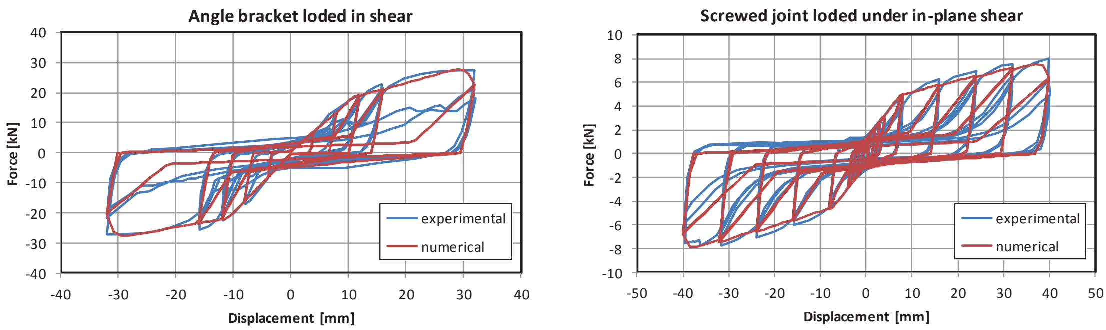


Figure 9. Experimental and analytical hysteresis curves comparisons

Table 4.1. Parameters of multilinear cyclic model of links

Trilinear model parameters		**L-stub**	**angle bracket**		**screwed joint**
		traction	traction	shear	shear
EI - Initial flexural rigidity	kN/mm	18.33	4.93	11.05	21.5
PCP - Cracking Force (positive)	kN	11.29	1.38	0.59	9.64
PYP - Yield Force (positive)	kN	35.73	7.6	18.68	32.65
UYP - Yield Displacement (positive)	mm	2.94	2.89	11	7.56
UUP - Ultimate Displacement (positive)	mm	53	25	33	41
3P - Post Yield Flexural Stiffness (positive) as ratio of EI	-	0.061	0.21	0.049	0.03388
PCN - Cracking Force (negative)	kN	-150	-40	-0.43	-6.69
PYN - Yield Force (negative)	kN	-150	-40.001	-21.91	-30.29
UYN - Yield Displacement (negative)	mm	-8.1833	-8.1136	-11	-7.3
UUN - Ultimate Displacement (negative)	mm	-100	-100	-33	-41
3N - Post Yield Flexural Stiffness (negative) as ratio of EI	-	0.999	0.999	0.034	0.04056
HC - Stiffness degrading parameter	-	2000	200	2000	2000
HBD - Ductility-based strength decay parameter	-	0.01	0.2	0.05	0.03
HBE - Hysteretic energy-based strength decay parameter	-	0.01	0.2	0.03	0.06
HS - Slip parameter	-	0.65	0.25	0.3	0.37

In order to simulate the unilateral constraint given by foundation and by the floors on the vertical displacement of the panels, asymmetric springs, also described by the multilinear model, in parallel with a gap element have been adopted. The masses have been lumped in the nodes of the slab and a viscous damping of 5% was used. In order to assess the reliability of the numerical model of the building, the Kobe accelerogram with 0.82g peak ground acceleration was applied to the model. So the obtained results were compared in terms of displacements with the experimental results of shaking table tests (Table 4.2).

Table 4.2. Experimental and analitical model displacements comparisons

Max displacement		z-direction (mm)		x-direction (mm)					
		0NE	0SE	1NE	2NE	3NE	1SE	2SE	3SE
	test	10.65	7.39	26.00	51.50	58.90	29.50	56.10	62.20
Kobe 0.82g	model	9.89	6.74	24.10	49.20	57.40	27.80	53.30	60.00
	difference	7.1%	8.8%	7.3%	4.5%	2.5%	5.8%	5.0%	3.5%

The differences between the horizontal displacements of the numerical model and the ones recorded in the experimental tests do not exceed 7.3%, while the vertical hold-downs displacements do not exceed 8.8%.

4.2. Definition of Yield and Collapse Limit and Analysis of the Results

The yield limit of the building has been individuated by checking the achievement of the following condition:

$$F_{\max} \geq F_{y,\lim} \tag{4.2}$$

where F_{max} is the force acting in the most stressed XL-stub/Hold-Down and $F_{y,lim}$ is its yield force defined, according to Eurocode 3, in correspondence of 2/3 of the plastic resistance. The plastic resistance of the element has been conventionally determined by intersecting the monotonic force-displacement curve with a straight line with slope equal to $K_\varphi/3$, being K_φ the initial stiffness. In particular, starting from the experimental curves, for the XL-stub an elastic limit equal to 34.6 kN has been found.

The ultimate acceleration multiplier α_u has been evaluated in correspondence of the failure of the most stressed XL-stub or Hold-down, which leads to the collapse of the whole building. The collapse condition on the XL-stub has been defined by exploiting the fatigue life curve previously defined. It is assumed that the failure of the XL-stub arises when the level of damage becomes equal to one. In order to determine the level of damage through the Miner's rule, it has been necessary to apply a counting method in order to account for the randomness of the loading history. In particular, the "rainflow" counting method has been applied developing the following steps:

1. rearrangement of the loading history starting from the maximum plastic excursion experienced during the whole loading history;
2. coupling of the absolute maximum with the absolute minimum in order to form the cycle of maximum amplitude;
3. coupling of all relative maximum points with the closest minimum relative points in order to determine the cycles of lower amplitude.

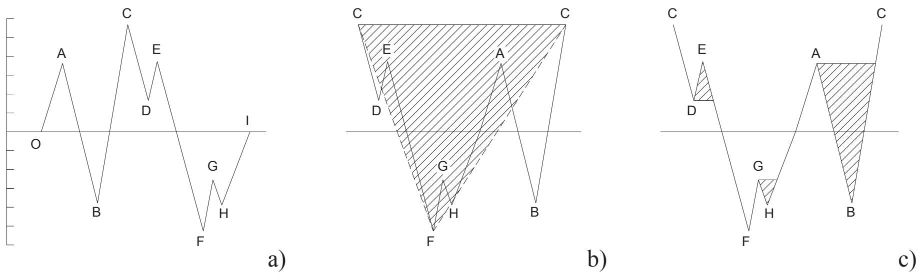


Figure 10. Rainflow counting method: a) random time history; b) rearranged time history and max amplitude loop (CF); c) other cycles (ED, GH, AB)

A first set of Incremental Dynamic Analyses has been carried out for increasing values of the multiplier of the non-dimensional accelerogram (α). The multiplier has been increased step by step by choosing an increase size of the PGA equal to 0.2. When one of the limits previously defined was exceeded (yielding or failure of an XL-stub), starting from the value of the multiplier immediately below the one individuated, a second set of analysis was carried out by using an increase size of the step equal to 0.05. In this way it has been possible to achieve a good accuracy in the determination of α_y and α_u.

Table 4.3. Analisys results

Earthquake	building with L-stubs			building with hold-downs		
	α_y	α_u	q	α_y	α_u	q
El Centro (19/5/40, Imperial Valley, N-S, 40.0s)	0.50	2.25	4.50	0.35	1.20	3.43
Kobe (16/1/95, JMA, N-S, 48.0s)	0.45	2.10	4.67	0.35	1.15	3.20
Kocaeli (17/8/99, Yapi Kredi, N-S, 85.80s)	0.45	2.20	4.89	0.35	1.43	4.09
Loma Prieta (18/10/89, Corralitos, E-W, 39.98s)	0.55	2.70	4.91	0.35	1.05	3.00
Nocera Umbra (27/7/97, Nocera, E-W, 13.7s)	0.45	3.00	6.67	0.35	1.60	4.57
Northridge (17/1/94, Newhall, E-W, 19.98s)	0.40	2.50	6.25	0.35	0.88	2.51

For the same accelerograms the values of the q-factor of the building with Hold-downs has been find in scientific literature (Ceccotti et al., 2006). It should be emphasized, however, that these values were obtained imposing the collapse condition of the building in correspondence of an Hold-down uplift of 25.5 mm, which corresponds to the maximum displacement observed in the test on shaking table with 0.90g Kobe accelerogram. Therefore, with reference to the Hold-downs the phenomenon of low-cycle fatigue has been neglected. In addition, in (Ceccotti et al., 2006) the value of α_y was fixed a priori as equal to design value of 0.35.

The comparison between the response of the building with hold-downs and with XL-stubs shows that the application of XL-stubs to timber panel buildings provides a significant increase in terms of behavior factor. In particular, the value of the calculated q-factor for the three-storey building with XL-stubs is not less than 4.50 and is, therefore, 1.5 times greater than the value equal to 3 recommended by (Ceccotti et al., 2007) for structures employing Hold-downs.

5. CONCLUSIONS

Cross laminated timber panel buildings are characterized by low energy dissipation capacity because the dissipative zones are only represented by the Hold-downs which connect the panels to the foundation or to the slabs. In this paper, the developed experimental program aimed to set-up an innovative connecting elements called XL-stub, has shown that the proposed connected element is able to significantly increase the energy dissipation capacity of the structure. Starting from the experimental results, a proper model of the hysteretic behavior of the XL-stub has been determined and IDA analyses of a three story building have been performed in order to evaluate the q-factor. The results of the analyses developed submitting the structure to six natural accelerograms show a minimum value of the q-factor equal to 4.5 for the proposed system, which is significantly greater of that evaluated considering common hold-down connecting elements.

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