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SEISMIC PERFORMANCE OF X-LAM BUILDINGS: THE ITALIAN SOFIE PROJECT

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ABSTRACT

In this paper the seismic performance of wooden buildings made of Cross-laminated-timber (XLAM) is illustrated as a part of an extensive research project undertaken in Trentino (Italy) to promote the use of XLAM for residential and non residential buildings. Strength, deformability and capability of dissipating energy are addressed. Design action reduction factor is also evaluated according to Seismic Eurocode format. Finally the amazing capability of "shape keeping" of such buildings under the design quake is highlighted.

Introduction

Buildings made of massive cross-laminated (X-lam) timber panels as wall and floor panels are becoming a stronger and economically valid alternative to traditional masonry or concrete buildings in Europe. Especially in seismic-prone countries as Italy, X-lam buildings are gaining more and more popularity among architects and customers. However, knowledge is limited about the earthquake behavior of such buildings although timber lends itself for earthquake applications due to its good weight-to-strength-ratio. Consequentially, a large research project, called SOFIE, was started to investigate, among other issues such as fire resistance, building physics and durability, the seismic behavior of X-lam buildings. Within this project, full-scale shaking table tests were carried out on a three-storey and a seven-storey specimens.

The product X-lam is made of spruce planks with a thickness of around 15 to 40mm assembled two-dimensionally and then spread with glue. On the glue line, the next layer of planks is assembled orthogonally to the lower layer. The result is a massive, dimension-stable plywood-type of plate material available in different total thicknesses according to different structural needs. With this very stiff and rigid product, multi-storey buildings can be easily achieved.

The comprehensive research project SOFIE is a cooperative project supported by the Trento Province, Italy and coordinated and conducted by the IVALSA-CNR (Trees and Timber Institute - Italian National Research Council). The shaking table tests of the project are carried out together with Japanese partners from Shizuoka University, Building Research Institute and the National Institute for Earth Science and Disaster Prevention, NIED. The tests on the seven-storey building were carried out on NIED's 3D 20x15m shaking table in Miki close to Kobe. It was the first time that a full-scale seven-storey building, of whatever material, was tested on a 3D shaking table.

One of the aims of SOFIE was to evaluate the earthquake performance of multi-storey X-lam buildings and to determine necessary parameters such as force reduction factors in order to design these buildings in earthquake regions. The earthquake project was divided in different

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research parts following a hierarchical structure; the tests started at material level and going over structural element level up to tests on full-scale 3D buildings:

- tests on connections and X-lam panels (bending tests);
- in-plane quasi-static reversed cyclic tests on wall panels with different connections and opening layouts and with different dimensions and amounts of vertical load;
- pseudo-dynamic tests on a one-storey specimen in 3 different opening layouts in the external walls parallel to the shaking direction and without vertical load;
- 1D-shaking table tests on a three-storey building of about 7m x 7m in plan and 10m of total height with 3 different openings (configuration A, B, C) on ground floor and 15 tons additional weight per storey;
- 3D-shaking table tests on a seven-storey building of about 7.5m x 13.5m in plane and 23.5m of height with 30 tons additional load per storey.

The following article will present the SOFIE project focusing on the shaking table tests and code implications. For more exhaustive information on cyclic and pseudodynamic test setup and results please refer to the literature (Ceccotti et al 2006, Lauriola et al 2006). Especially the evaluation of action reduction factors or behavior factors as they are called in Europe will be addressed. Behavior factors can be "used for design purposes to reduce the forces obtained from a linear analysis, in order to account for the non-linear response of a structure, associated with the material, the structural system and the design procedures" (Eurocode 8).

SOFIE Buildings

Preparative Tests

The monotonic and cyclic tests on wall panels were carried out for two main reasons. First of all, first information was necessary to assess the cyclic behavior of wall elements, especially their racking behavior. The energy dissipation capacity of these important structural elements needed to be established and the connections had to be calibrated in order to get a ductile failure mode. No brittle failures of the connections should occur, as in this construction method with very rigid panels, all ductile behavior is concentrated in the connections. The second reason was the necessity to get input parameters for numerical modelling of X-lam structures under earthquake loading (for the modelling, please refer to Ceccotti 2008).

Consequently, in-plane monotonic and cyclic tests were carried out as described in the literature (Ceccotti et al 2006). The tests were carried out on 2.95m x 2.95m wall panels with different vertical loading. Various wall assemblies were tested; from ground floor panels to upper floor panels and with and without openings.

The test outcomes confirmed the importance of the connections. The layout and design of the connections is influencing strongly the overall behavior of the structural system. The X-lam panels behaved almost completely rigid with no deformations. The found viscous elastic damping rate ranged from 11% to 18.5% which indicated a good dissipative behavior with broad hysteresis loops. The lateral load capacity was between 39kN/m and 48kN/m for the ground floor panels with a secant stiffness between 7.3kN/mm and 6.0kN/mm. The tests on wall panels hence showed, that the X-lam system had a high stiffness, but still good ductile and dissipating performances making it very promising for seismic purposes.

Geometry

The X-lam system is a modular system where all X-lam panels are pre-cut in factory, transported on site and then assembled storey by storey by installing the wall panels inclusive all connections before putting the floor panels on top of the walls. This platform-frame principle is not the only possible method; X-lam wall panels could also reach over multiple storeys. Two representative photos of this construction system are shown in Figs. 1 and 2. The single panels are usually connected by commercially available steel connectors and nails and/or self-drilling screws.

Figure 1. Typical X-lam building
(Photo Rasom wood technology)

Figure 2. Three-storey SOFIE building

As can also be seen in Fig. 2, both SOFIE buildings were entirely made of X-lam panels made from spruce coming from certified forests in the Trentino region in Northern Italy. The panel thickness of the three-storey building was 85mm for the wall panels and 142mm for the floor panels whereas the thickness of the floor panels remained the same for the seven-storey building with wall thicknesses of 142mm for the two lower storeys, 125mm for storeys 2 and 3 and 85mm for storeys 4,5 and 6. The wall panel thickness was thus varied according to the structural needs; however, inner and outer walls were of the same thickness.

In Fig. 3, the geometry of the three-storey building in configuration C can be seen. This building was tested with three different ground floor openings:

- Configuration A with three openings of 1.20m x 2.20m;
- Configuration B with three openings of 2.25m x 2.20m;
- Configuration C with one opening of 4.00m x 2.20m in one outer wall and two other openings of 2.25m x 2.20m (see Fig. 3).

The seven-storey building was tested in only one configuration whose geometry can be seen in Figs. 4 and 5.

Figure 3. Plan views and elevations of three-storey building in configuration C

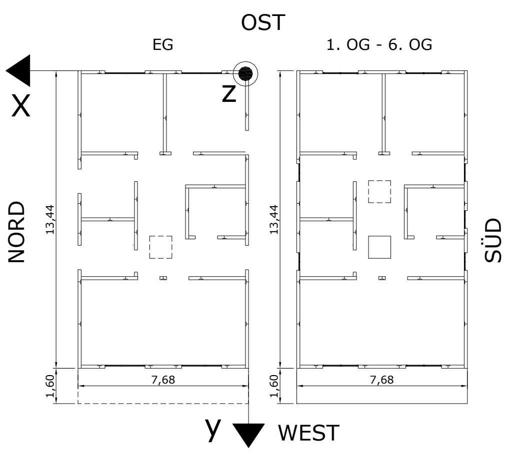


Figure 4. Plan views of seven-storey building

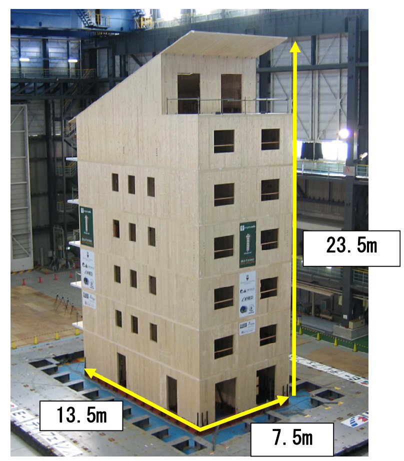
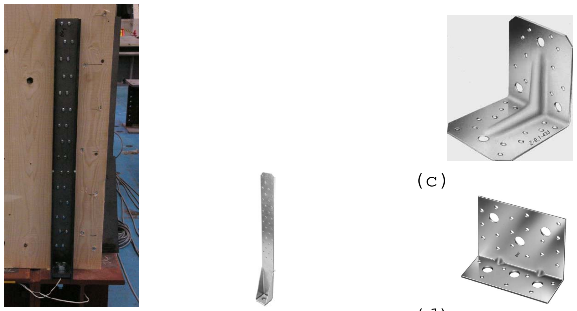


Figure 5. Seven-storey building

Connections

The connections were calibrated during the monotonic and cyclic tests as already stated. Not only the connection type, but also the number of nails were determined in order to get a ductile failure mode of the connections. The horizontal forces were taken by the in regular distances arranged shear connectors (Fig. 6, c+d), which connect the floor panels (resp. the foundation) with the upper walls. The shear connectors were fastened with annular ringed shank nails. Hold-down anchors (Fig. 6, a+b) were arranged in the building corners and at the door openings to take the high uplifting forces resulting from the high horizontal seismic shear. Simpson HTT22-hold-down anchors as shown in Fig. 6b were chosen for the three-storey building. The HTT22 were fastened to the wall panels with annular ringed shank nails. For the seven-storey building, these hold-down anchors had to be replaced by specially fabricated 'IVALSA'-hold-downs as shown in Fig. 6a. The reason for this were the considerably higher uplift forces in the seven-storey building which could not be taken by the weaker HTT22. The special ‘IVALSA hold-downs were fastened to the wall panels with lag screws. The uplift-connection between the storeys is shown in Fig. 7 and consisted of two hold-down anchors connected through the floor slab with a rod.



(a) (b) (c) (d)

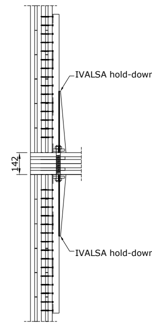


Figure 6. (a) ‘IVALSA’ anchor of 7-storey-building
(b) Simpson HTT22-hold-down
(c) Shear connector upper storeys
(d) Shear connector ground floor

Figure 7. Interstorey hold-down connection

The in-plane wall-to-wall connection was made with notches in the two adjacent panels covered by an LVL-strip fastened with self-drilling screws (Sandhaas et al 2009). All other connections such as the connections of the floor slabs and the connection of the floor slabs on the lower walls were done with self-drilling screws.

Generally, a hierarchical system of designing the connections was used. Critical connections like the connection of the perpendicular wall panels in the building corners were designed to be stiff as well as the connection of the floor slabs to the lower walls. These connections should not fail during an earthquake. The ductility and energy dissipation of the X-lam buildings were assigned to the uplift connections (hold-downs), the shear connectors and the in-plane wall-to-wall connection.

Additional Load

Figure 8. Additional load on floor

In Europe, X-lam buildings are usually very heavy. Due to acoustical insulation, an extra layer of sand and a floating floor is added. Furthermore, the tested buildings was not finished, but consisted only of the wooden elements. Therefore, additional loading has to account for the heavy floors, insulation and other wall finishing as well as for 30% percent of imposed loads as required by (Eurocode 0). The three-storey building was additionally loaded as shown in Fig. 8 with 15t per storey; the seven-storey building with 30t per storey. In total, the three-storey building weighed 465kN whereof 30kN additional load. The seven-storey building had a total weight of 2840kN whereof 1500kN additional loading.

Testing Sequence

Three-Storey Building

The three-storey building was tested on the 1D shaking table of NIED in Tsukuba, Japan with three different earthquakes: Kobe JMA N-S, El Centro and the Italian earthquake of Nocera Umbra. The peak ground accelerations (PGAs) of the earthquakes were sequentially increased from 0.15g to 0.5g for all three configurations. Only configuration C was tested up to near collapse increasing further the PGAs.

Seven-Storey Building

The seven-storey building was tested on the 3D shaking table of NIED in Miki, Japan. First 1D earthquakes were applied, then 2D earthquakes and the last 4 earthquakes were 3D earthquakes using all three earthquake components N-S, E-W, U-D. Table 1 shows the test sequence and in Figs. 4 and 5, the definition of the directions can be seen.

Table 1. Test sequence of seven-storey building

test number	input	direction	dimension	intensity	PGA in x	PGA in y
1	step	X, Y	2D		0.3g	0.3g
2	Nocera Umbra E-W	Y	1D	70%	-	0.35g
3	Nocera Umbra E-W	Y	1D	100%	-	0.5g
4	JMA Kobe N-S	Y	1D	60%	-	0.5g
5	JMA Kobe E-W	X	1D	50%	0.3g	-
6	step	X, Y	2D	-	0.3g	0.3g
7	JMA Kobe N-S	Y	1D	100%	-	0.82g
8	step	X, Y	2D	-	0.3g	0.3g
9	JMA Kobe E-W	X	1D	100%	0.6g	
10	step	X, Y	2D	-	0.3g	0.3g
11	step	X, Y	2D	-	0.3g	0.3g
12	JMA Kobe interrupted	X, Y, Z	3D	100%	0.6g	0.82g
13	step	X, Y	2D	-	0.3g	0.3g
14	step	X, Y	2D		0.3g	0.3g
15	Kashiwazaki R1	X, Y, Z	3D	50%	0.155g	0.34g
16	step	X, Y	2D		0.3g	0.3g
17	step	X, Y	2D		0.3g	0.3g
18	JMA Kobe	X, Y, Z	3D	100%	0.6g	0.82g
19	step	X, Y	2D		0.3g	0.3g
20	step	X, Y	2D		0.3g	0.3g
21	Kashiwazaki R1	X, Y, Z	3D	100%	0.311g	0.68g
22	step	X, Y	2D		0.3g	0.3g

Instrumentation

The instrumentation of these full-scale dynamic tests was rather complex; for the seven-storey 3D test, a total of 266 channels were used. The main measuring systems are presented in Figs. 9 to 12:

- Interstorey drift, measured from lower to upper floor slab (Fig. 9);
- Uplift at corner hold-downs (Fig. 10);

- Deformation of the in-plane wall-to-wall connection (Fig. 11);
- Accelerations in the different storeys.

In the seven-storey building, other deformations such as the slip between floor panels or the deformation of the connection with inclined self-drilling screws between floor slabs and lower walls have not been explicitly measured, as they were designed to be very stiff and no deformation was expected. The previous test on the three-storey building, where these values had been measured, confirmed this assumption and the designing procedure.

Figure 9. Interstorey drift

Figure 10. Uplift

Figure 11. In-plane wall-to-wall joint

Figure 12. Accelerometers

Near-Collapse Criterion

The near-collapse criterion of both shaking table tests was defined as the failure in one or more hold-down anchors (broken nails, screws, bolts or steel plate).

Design of the Three-Storey Building

As the building is regular in plan and elevation, the simplified method according to (Eurocode 8) chapter 4.3.3.2 could be used, which works with equivalent horizontal forces reduced by a behavior factor. First, the seismic base shear was determined with:

$$F_b = \gamma_I \cdot m \cdot a_g \cdot S \cdot \frac{2.5}{q} \qquad (1)$$

where
γ_I = importance factor of the building (taken equal to 1 as for residential buildings);
m = total mass of the building;
a_g = design ground acceleration;
S = soil factor;
q = action reduction factor a.k.a. behavior factor to reduce design forces obtained from a linear calculation in order to account for the capability to dissipate energy.
The seismic base shear calculated according to Eq. 1 is then distributed on the storeys by Eq.2:

$$F_i = F_b \frac{z_i \cdot m_i}{\sum z_j \cdot m_j} \qquad (2)$$

where z_i, z_j are the heights of the masses m_i, m_j above the level of application of seismic action.

The value for a_g was taken to 0.35g, which is the highest PGA for the Italian territory. As nothing was yet known about a behavior factor q for X-lam buildings and in order to determine a preliminary q-factor after reaching the near-collapse criterion, a value of q=1.0, purely elastic building response, was chosen.

The connections were then designed with the horizontal forces listed in Table 2. The number of connectors, nails and screws was determined. This number was decreasing on the upper floors as there the shear forces are smaller.

Table 2. Seismic forces for three-storey building

Mass of the building

roof		45	kN
floor 2		210	kN
floor 1		210	kN
	TOT	**465**	**kN**

seismic forces

seismic base shear			
Zone 1; a_g =		0.35	
T1		0.20	
Soil class B S=		1.25	
q		1	
$\mathbf{F_b}$ = 2,5*(W*S*a_g)/q		**509**	**kN**
distribution on storeys			
height			
	Zr (roof) =	9.40	m
	Z2 (floor 2) =	6.18	m
	Z1 (floor 1) =	3.09	m
horizontal forces per storey			
	Fr =	91	kN
	F2 =	279	kN
	F1 =	139	kN
shear per storey			
	Tr =	91	kN
	T2 =	370	kN
	T1 =	509	kN

Design of Seven-Storey Building

The seven-storey building was designed accordingly with the following PGAs:

- a_g = 0.82g when shaking along Y (long direction);
- a_g =0.6g when shaking along the short direction X.

The other changed parameter in comparison to the design of the three-storey building was the behavior factor q. This q-factor resulted to q = 3 after the shaking table tests on the three-storey building as will be presented in the following chapter. Therefore, a behavior factor of q=3 was chosen to calculate the seismic base shear of the seven-storey building. Nevertheless an importance factor γ_I =1.5 was chosen this time as for strategic buildings, i.e. buildings that must be completely operational even after a destructive quake (Hospitals, Civil protection headquarters, etc.).

Results

Three-Storey Building

The results of the three-storey building are summarized for configuration C in Table 3. Before the listed sequence of major earthquakes, no damage was observed in configurations A, B and C. Even after reaching the near collapse criterion, the building has kept its shape with no residual displacements and without major reparations. The near-collapse criterion was reached after the earthquake of Nocera Umbra with a PGA of 1.2g (original PGA_{max} of Nocera Umbra was 0.5g) as can be seen in Fig. 13. Most nails of ground floor hold-downs went broken

Figure 13. Hold-down failure after Nocera Umbra 1.2g

With this result, an evaluation on the behavior factor q for the tested building could be done. The strategy was as follows:

- Design the structure using q=1 according to the seismic code for a given design $PGA_{u,code}$ (0.35g - which is the design ground acceleration corresponding to the most hazardous seismic zone of Italy);
- Define as "near-collapse" criterion the failure in one or more hold-downs;
- Analyse the test results and calculate q as the ratio between the $PGA_{u,eff}$ value that caused the near-collapse of the building and the design value of the $PGA_{u,code}$.

Therefore, being the design ground acceleration $PGA_{u,code}$ equal to 0.35g, by applying the quoted procedure, the q value is:

$$q = \frac{1.20}{0.35} = 3.4 \tag{2}$$

Table 3. Results of shaking table tests for configuration C en terms of observed damage

Record	**PGA [g]**	**Restoring intervention (before the test)**	**Observed damage (after the test)**
Nocera Umbra	0.50	Tightening of holdown anchor bolts	None
El Centro	0.50	Tightening of holdown anchor bolts. Replacing of screws in vertical joints between panel	None
Kobe	0.50	Idem	None
Kobe	0.80	Idem	Slight deformation of screws in vertical joints between panels
Kobe	0.50	Idem	None
Kobe	0.50	Tightening of holdown anchor bolts	None
Kobe	0.80	Replacing of holdown anchors and tightening of bolts. Replacing of screws in vertical joints between panel	Slight deformation of screws in vertical joints between panels
Nocera Umbra	1.20	Tightening of holdown anchor bolts. Replacing of screws in vertical joints between panel	Holdown failure and deformation of screws in vertical joints between panels

Seven-Storey Building

As already stated and based on the results of the shaking table tests on a three-storey X-lam building, the seven-storey building was designed with a behavior factor of q=3 and a γ_I =1.5.

After the whole series of earthquakes as listed in Table 1 and after not even reaching a real near-collapse state, no residual displacement could be measured on final tests. Table 4 lists the observed damage after the also listed tests and the subsequent repair. Other measured deformations were a maximum uplift during JMA Kobe 3D 100%, test number 18, at ground floor level of 13.19mm which is smaller than the value resulting from cyclic tests at which the special 'IVALSA'-hold-down failed and which resulted to 30mm. The maximum interstorey drift during JMA Kobe 3D 100% resulted to 67mm between first and second storey – again smaller than the value of 80mm at which the connections failed during the cyclic test.

Table 4. Test results on seven-storey building in terms of damage and repair

Test n.	input
16	step
Damage	No damage on hold-downs, loose hold-down bolts, pulling-out of nails in steel angles – especially on upper storeys.
Repair	Hold-down bolts tightened, ringed nails in steel angles driven back and some nails added.
19	step
Damage	Hold-downs in 2F and 3F (between floor slabs 1F/2F and 2F/3F) damaged but not failed (screws pulled out and bent, not broken), pulling-out of nails.
Repair	Hold-down bolts tightened (nothing else was done, pulled-out and bent screws were not changed), nails driven back.
22	step
damage	Often: pulling-out of ringed nails in steel angles, no further damage observed

Conclusions

The test outcomes indicate that XLAM buildings are very feasible for well performing construction in earthquake zones and the proposed behavior factor of q=3 is a reasonable value for seismic design of such buildings. At a PGA of 0.82g no significant damage occurred in the seven storey building designed with an importance factor γ_I =1.5 as for strategic buildings: not only did the building survive the single devastating earthquake JMA Kobe from 1995, but it also resisted a whole series of earthquakes in 1D and 3D keeping its shape and remaining fully operational.

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