Earthquake resistance of multi-storey massive timber buildings

Erdbebensicherheit von mehrgeschossigen Massivholzbauten

Sécurité au risque sismique en constructions pluriétage en bois massif

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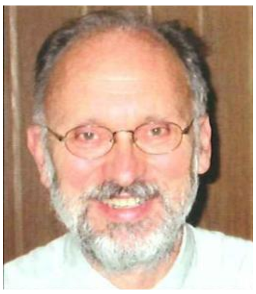

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1. Introduction

Structural design of multi-storey massive timber buildings must include verifications of their lateral stiffness and stability under horizontal loading, e.g. wind loading. A special case of horizontal loading is earthquakes whose impact on buildings is dependent on the soil conditions, different seismic parameters such as acceleration values and the building itself through its natural frequency. In France, seismic zonation maps are defined which give geologically possible peak ground acceleration (PGA) values needed to calculate seismic shear forces acting on structures (see Figure 1 [1]). However, earthquakes not only cause high horizontal loads, but seismic loading is also cyclic and dynamic. The more complex a structure is, the more difficult it is to evaluate the structure's response under seismic loading.

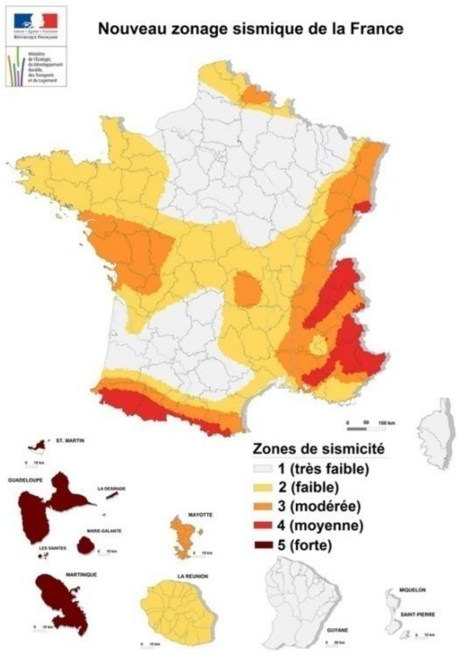


Figure 1: Seismic zonation France [1]

Therefore, good earthquake design is based on the best possible compliance of construction principles such as structural simplicity, regularity, uniformity and symmetry of structures or redundancy through creation of alternative loading paths and load redistribution. If these principles are maintained, structures can be designed with simplified methods without the need of undertaking complex modal analyses and dynamic calculations. Furthermore, earthquake behaviour of structurally simple buildings is easier to understand which leads to more reliable design. Design concepts and construction principles are defined in modern seismic codes (e.g. EC8 [2]). Comprehensive descriptions are available in the introducing literature (e.g. in [3]).

However, in order to carry out a reliable seismic design, certain parameters are necessary which capture the construction principle of the structure to be designed. These seismic design parameters must be established in standards and are evaluated by research projects. The evolution of such a research project and the implementation of seismic parameters in standards are shown in this article by presenting an example project. The example project shows the suitability of massive multi-storey timber buildings in earthquake-prone areas.

2. Project SOFIE

2.1. Basic Idea

The SOFIE project was a comprehensive research project carried out by the Trees and Timber Institute of the Italian National Research Council, CNR-IVALSA, and was financed by the Autonomous Province of Trento. The scope of the project was to investigate multi-storey massive timber buildings made from cross-laminated timber (CLT) panels by considering all technical aspects; from fire resistance over acoustics to durability and building physics.

A main objective was to evaluate the earthquake behaviour of massive CLT timber buildings. Italy is highly seismic, the whole national territory is seismic region and seismic design is hence indispensable. However, no information was available about the seismic resistance of CLT buildings. No structural details or design parameters for CLT buildings are given in the European earthquake standard EC8.

Therefore, extensive testing series have been carried out in order to classify CLT constructions and to establish seismic design factors.
The earthquake project was divided in different research parts following a hierarchical structure; the tests started at material level over structural element level up to tests on full-scale buildings:

- Monotonic and cyclic tests on wall elements in order to evaluate the load carrying capacity in-plane; different connections, openings and vertical loads were considered;
- Pseudo-dynamic tests on a single-storey CLT specimen, 7x7 m in plane, three different openings and no vertical loads;
- 1D shaking table tests on a three-storey building of about 7x7 m in plan and 10 m of total height with 3 different ground floor openings and 15 tonnes additional weight per storey;
- 3D shaking table tests on a seven-storey building of about 7.5x13.5 m in plan and 23.5 m of total height with 30 tonnes additional weight per storey.

The first two testing series were needed to calibrate the connections of the CLT buildings. No brittle failure modes should occur and all connections should be ductile and energy-dissipating. Furthermore, numerical models can be developed based on test results of wall elements. Numerical models are necessary to vary building geometries and earthquakes in order to develop reliable seismic design parameters which can be inserted in EC8. However, a first indicative design parameter can be derived based on full-scale shaking table tests which is valid for the tested geometry under the tested loading.
The first two testing series are not presented here, please refer to literature for more information [4, 5, 6].

2.2. SOFIE Buildings

Geometry

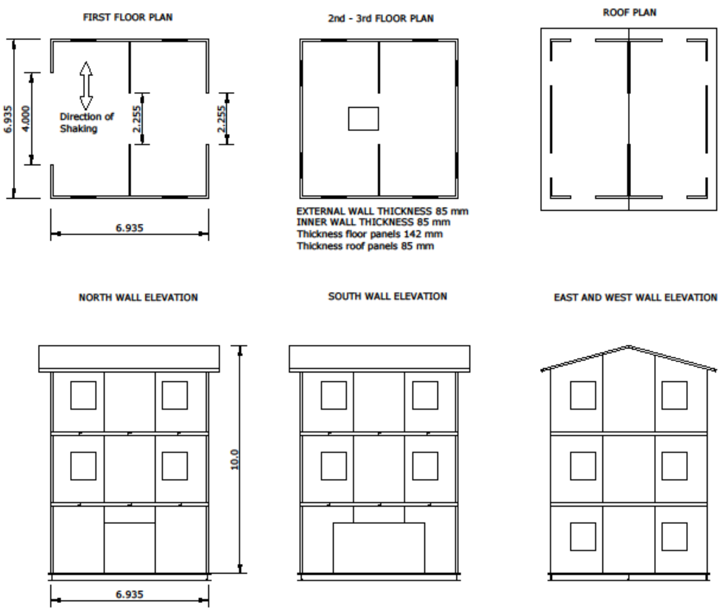


Figure 2: Three-storey SOFIE building in configuration C

Before presenting the tests, test results and design approaches in seismic standards, the two tested SOFIE buildings and some important construction details are presented here.
Both buildings are pure CLT buildings. Walls and floors are made of CLT panels with different thicknesses. Connections are standard timber connections using steel anchors, screws and nails.

The three-storey building was tested in three configurations with different opening sizes on ground floor level. Configuration A had three openings with a width of 1.20 m each. The three openings were broadened in configuration B to a width of 2.25 m each. Configuration C finally is shown in

Figure 2 and had an asymmetric big opening of 4.00 m in one external wall. The height of the openings was not changed and was 2.20 m for all configurations. The thickness of the CLT wall elements was 85 mm and 142 mm for the floor elements.
The floor element thickness of the seven-storey building remained the same with 142 mm. However, the thickness of the wall elements was varying per storey due to different structural needs. Inner and outer walls were of the same thickness. On ground and first floor, the wall elements were 142 mm thick, 125 mm in storeys two and three and 85 mm in the upper floors. The plan views of the 7.5x13.5 mm building are shown in Figure 3 whereas a rendering is shown in Figure 4.

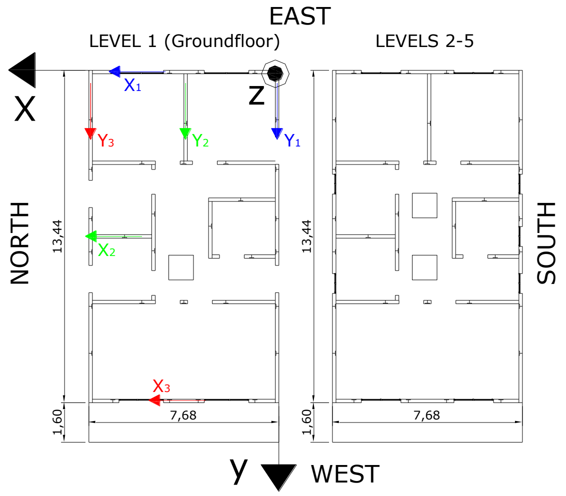


Figure 3: Plan view of the seven-storey building

Figure 4: Seven-storey SOFIE building, Rendering

Additional Loads

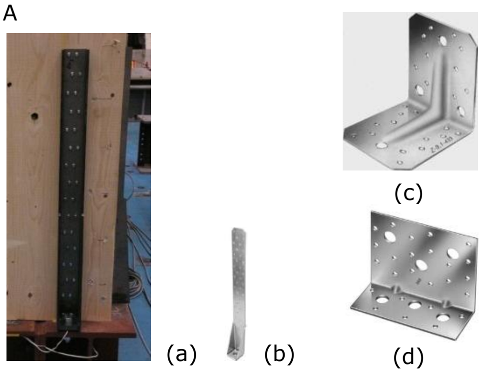
Figure 5: Additional Loads

A finished CLT building is usually quite heavy. Due to acoustic insulation, the floor build-up consists of an extra layer sand and a floating floor. The wall panels are usually covered with an insulating layer and an installation level with gypsum plaster boards (fire resistance). Furthermore, according to EC0 [7], 30% of the imposed loads must be considered. On the shaking table, only the „building shell" could be tested. The additional loading was simulated with steel plates as shown in Figure 5 in order to carry out a seismic test under realistic conditions. Especially for dynamic loading, it is essential to simulate the correct masses of the tested structures.
The three-storey building was loaded with 15 tonnes additional mass per storey (total of 30 tonnes). 30 tonnes per storey were used for the seven-storey building (total of 150 tonnes). The total mass of the buildings was 47 tonnes of the three-storey building and 285 tonnes of the seven-storey building.

Connections

All connections were carried out with standard connectors. The shear forces were transmitted by steel angles placed at regular intervals that connected the floor panels with the upper wall panels (Figure 6; c+d). Hold-downs were used in the building corners and at openings to accommodate the high uplift forces that can be generated by high horizontal seismic loads (Figure 6, a+b). For the three-storey building, Simpson HTT22 hold-downs were used (Figure 6b) which were replaced by specially fabricated IVALSA hold-downs in the seven-storey building (Figure 6a). The reason for this replacement was the consid erably higher uplift forces in the slender seven-storey building that could not be trans ferred by the HTT22. The uplift connection between the storeys is shown in Figure 7 and consisted of two hold-down anchors connected through the floor slab with a rod. The in-plane wall-to-wall connection was made with notches in the two adjacent panels cov-

A



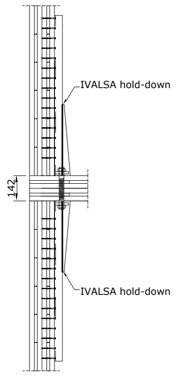


Figure 6: (a) IVALSA hold-down
(b) Simpson HTT22
(c) steel angle upper storeys
(d) steel angle ground floor

Figure 7: Connection intermediate storey

ered by an LVL-strip fastened with self-drilling screws [8]. 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 (with self-drilling screws) of the perpendicular wall panels in the building corners were designed to be stiff just as the connection of the floor slabs to the lower walls. These critical connections should not fail during an earthquake as their failure and considerable deformation could cause collapse. The ductility and energy dissipation of the X-lam buildings were assigned to the uplift connections (hold-downs), the shear connectors (steel angles fixing upper walls to floor slabs) and the in-plane wall-to-wall connection. Furthermore, the number of screws and nails differed in the single storeys according to the changing seismic shear forces.

2.3. Design

The simplified lateral force method as defined in EC8 [2] can be used for seismic design if certain requirements are met. For instance, regularity in plan and elevation is such a requirement. Horizontal bracings running without interruption from their foundations to the top of the building are structural examples that guarantee regularity in elevation. As already stated in the introduction, structural simplicity, symmetry and regularity of the buildings reduce the calculation needs considerably.

The simplified method according to EC8 is shortly presented here for the three-storey SOFIE building:

The seismic base shear F_b is calculated as follows:

$$F_b(T_1) = S_d(T_1) \times m \tag{1}$$

with S_d = ordinate of the design response spectrum at period T_1 and m = mass of the building.

The period of the three-storey building in the considered direction was T_1=0.20s, the ordinate follows to:

$$S_d(T_1) = a_g \times S \times \frac{2.5}{q} \tag{2}$$

with a_g = design peak ground acceleration value of an earthquake, often expressed as a percentage of gravity g=9.81m/s^2; e.g.: 0.35g=35% of g;
S = soil factor;
q = behaviour factor.

The seismic base shear is then distributed in horizontal forces per storey:

$$F_i = F_b \frac{z_i \cdot m_i}{\Sigma z_j \cdot m_j} \tag{3}$$

with z_i, z_j the heights of the masses m_i, m_j above the level where F_b acts (usually foundations).

Afterwards, a „static" design applying the evaluated horizontal forces can be carried out. Soil parameters, peak ground accelerations and design response spectra are defined in seismic standards.

The relevant design parameter that is of interest for a timber engineer is the behaviour factor q, Equation (2). In order to avoid complicated nonlinear calculations, a linear verification is carried out. The applicable seismic forces are reduced by the behaviour factor q in order to account for the nonlinear response of the structure, associated with the material, the structural system and the design procedures [2]. This q-factor has to be chosen by the earthquake engineer for every building. In other words, the introduction of the behaviour factor q simplifies the design calculations considerably as instead of complex nonlinear calculations, linear verifications can be carried out with reduced forces that account for the ductility and energy dissipation of the building. The design of the three-storey SOFIE building from

Figure 2 has been carried out with the parameters for Italy. The design peak ground acceleration has been a_g=0.35g, the highest PGA in Italy (in mainland France, aléa moyen a_g = 3 m/s^2 = 0.3g), and the soil factor was S = 1.25. The initial design assumed a purely elastic building with no energy dissipation. This means
that a q-factor q = 1.0 was used. If for instance, the q-factor would have been assumed to be q = 2.0, then the seismic base shear would have been half the value shown in Table 1. With the evaluated horizontal seismic forces as given in Table 1, the building and the connections have been designed. Therefore, it is the behaviour factor q that has to be determined in order to be able to design CLT buildings in seismic regions.

Table 1: Seismic forces of 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

3. Shaking Table Tests

For the shaking table tests, FSC/PEFC certified spruce was transported from the Trentino region to Germany where the CLT panels were produced. Afterwards, the panels and the connectors were transported to Japan. Only one building per test (three and seven storeys) was produced. The two buildings have been subjected to a whole series of earthquakes

and, if necessary, repaired in between. The chosen earthquakes were the Japanese Kobe earthquake from 1995 (JMA Kobe), an Italian earthquake (Nocera Umbra 1997), the El Centro quake as a reference earthquake for research (only three-storey building) and the Kashiwazaki earthquake from July 2007 that occurred a couple of weeks before the tests (only seven-storey building). All earthquakes were applied with small peak ground accelerations that were gradually increased in the course of the testing series. The single earthquakes were followed by a so-called step input that subjected the buildings to free vibrations. With a step input, the development of the eigenfrequencies of the buildings could be measured and therewith the damage induced by the earthquakes.
Below, the original peak ground acceleration (PGA) values of the chosen earthquakes are given:

- JMA Kobe: North-South 0.82g, East-West 0.6g, Up-Down 0.34g (three-storey building only N-S), magnitude 7.2 on Richter scale;
- El Centro: 0.3g, magnitude 6.7 on Richter scale;
- Nocera Umbra: 0.5g, magnitude 5.8 on Richter scale;
- Kashiwazaki R1: North-South 0.68g, East-West 0.311g, Up-Down 0.408g (seven-storey building), magnitude 6.8 on Richter scale.

3.1. Method to Determine Behaviour Factor q

An important concept in modern force-based seismic standards is the behaviour factor q. As already stated, the q-factor reflects the capacity of buildings to dissipate energy through nonlinear behaviour and thus to survive destructive seismic events without collapse (loss of lives) – the so-called „near-collapse" state. The q-factor is determined by developing a numerical model that is able to simulate the nonlinear response of structures (different geometries and masses) subjected to different earthquakes. However, such a numerical model is difficult to verify without experimental data. Therefore, for every „new" construction typology that should be inserted in seismic standards, tests must be carried out. The experimental determination of the behaviour factor q is done as follows:

- Design the structure using q = 1 according to the seismic code for a given design peak ground acceleration a_g (here 0.35g);
- Define a near-collapse criterion, here the failure in hold-down anchors (one or more) and increase the PGA until near-collapse is reached, $a_{g,test}$;
- Analyse the test results and calculate q as the ratio between the value $a_{g,test}$ that caused the near-collapse of the building and the design value a_g.

This experimental approach gives an initial value for the behavior factor q that is valid only for the tested building and the earthquake which caused the near-collapse state. More information about design and determination of behavior factor q can be taken from literature [9, 10]. However, an experimental approach gives a qualitative measurement of the earthquake behaviour of a certain construction typology.

3.2. Measuring Techniques

A major challenge of full-scale shaking table tests is the question, which values should be measured and how. The four main measured values are listed below:

- Interstorey drift, measured from lower to upper floor slab (Figure 8);
- Uplift at corner hold-downs (Figure 9);
- Relative deformation of the in-plane wall-to-wall connection (Figure 10);
- Accelerations in the different storeys (Figure 11).

The first three connections are designed to behave in a ductile mode and to dissipate energy. The displacements of all other connections (i.e. between single floor slabs or at wall corners) was designed to be small, no energy should be dissipated in these important connections for structural safety. As already stated, this hierarchical design approach of the connections should avoid failure in connections that could trigger collapse. However, also there transducers were used to control the displacements and to verify that indeed only very small displacements occurred as defined in the design.


Figure 8: Interstorey drift


Figure 9: Uplift


Figure 10: Relative slip in plane wall-to wall connection


Figure 11: Accelerometers

3.3. Testing Series and Results Three-Storey Building

The three-storey SOFIE building was tested in three configurations (

Figure 2). The near-collapse criterion could only be reached in the last asymmetric configuration C as otherwise, the damage would have been too big already in configurations A and B and it would not have been possible to continue testing. The near-collapse criterion of the SOFIE criterion was the failure of one or more hold-downs.

The tests were carried out in July 2006 on the shaking table of the 'National Institute for Earth Science and Disaster Prevention' (NIED) in Tsukuba, Japan. The shaking table in Tsukuba is one-dimensional, it can be moved only in one horizontal direction – the earthquake direction is indicated in

Figure 2. Table 2 lists the observed damage for all earthquakes with a PGA bigger than 0.5g in configuration C. Before testing configuration C, also configurations A and B were already subjected to all three earthquakes with a_g = 0.15g und a_g = 0.5g. Also configuration C was subjected to smaller earthquakes with a PGA of 0.15g – a total of 15 earthquakes were applied before configuration C was tested up to near-collapse. Until 0.15g, no damage was observed in any configuration (no change of eigenfrequency) and the observed damage at 0.5g was small and repairable.

Table 2: Results for configuration C and earthquakes from 0.5g

Record	PGA [g]	Restoring intervention (before the test)	Observed damage (after the test)
Nocera Umbra	0.50	Tightening of hold-down anchor bolts	None
El Centro	0.50	Tightening of hold-down anchor bolts. Replacing of screws in vertical joints between panel	None
JMA Kobe	0.50	Idem	None
JMA Kobe	0.80	Idem	Slight deformation of screws in vertical joints between panels
JMA Kobe	0.50	Idem	None
JMA Kobe	0.50	Tightening of hold-down anchor bolts	None
JMA Kobe	0.80	Replacing of hold-down 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 hold-down anchor bolts. Replacing of screws in vertical joints between panel	**Hold-down failure** (see Figure 12) and deformation of screws in vertical joints between panels

The near-collapse criterion was reached during the Nocera Umbra quake with $a_{g,test}$ = 1.20g as can be seen in Table 2. The hold-down failure is shown in Figure 12. It must be emphasised that the three-storey building survived a series of 12 destructive earthquakes with peak ground accelerations of 0.5g and more without any major restoring interventions. In reality, the building has to survive one single of these earthquakes without collapsing. Even after Nocera Umbra 1.20g and hold-down failure, the building did not collapse and remained standing without permanent deformations. No influence of the asymmetric opening could be observed.

Figure 12: Hold-down failure after Nocera Umbra 1.20g

3.4. Determination of Behaviour Factor q

The design peak ground acceleration was a_g = 0.35g. The near-collapse criterion was met at a peak ground acceleration of $a_{g,test}$ = 1.20g. If now the above described method is used, the q-factor can be determined to:

$$q = \frac{a_{g,test}}{a_g} = \frac{1.20}{0.35} = 3.4 \qquad (4)$$

The evaluated q-factor is valid only for the tested configuration and the earthquake of Nocera Umbra. To derive a general q-factor for CLT buildings, a numerical model has been developed [9] whose simulations with different earthquakes resulted in q-factors between 3.0 and 4.57. However, still different geometries and building masses must be simulated in order to derive a final reliable behaviour factor q.
However, a value of q = 3.4 is a good indication. Together with the test results it can be concluded that CLT buildings show a good seismic resistance and that they are an adequate construction technique for earthquake-prone regions. More conclusions can be drawn after the shaking table test on the seven-storey building.

3.5. Testing Series and Results Seven-Storey Building

After the successful test series with a three-storey CLT building, a seven–storey CLT building (Figure 4) was tested on the big shaking table of the NIED in Kobe, Japan, in October 2007 – seven storeys because a higher building would not have fitted in the laboratory. The 15x20 m shaking table is three-dimensional. Therefore, all three spatial components of an earthquake, North-South, East-West and Up-Down, can be applied on a structure.
The design of the seven-storey building was not carried out with q = 1, but with q = 3 instead. This value of 3 resulted from the test series on the three-storey building.

Table 3: Testing series seven-storey building

input	PGA		
	in x	in y	in z
step 2D	0.3g	0.3g	-
Nocera Umbra O-W 1D 70%	-	0.35g	-
Nocera Umbra O-W 1D 100%	-	0.5g	-
JMA Kobe N-S 1D 60%	-	0.5g	-
JMA Kobe O-W 1D 50%	0.3g	-	-
step 2D	0.3g	0.3g	-
JMA Kobe N-S 1D 100%	-	0.82g	-
step 2D	0.3g	0.3g	-
JMA Kobe O-W 1D 100%	0.6g		-
step 2D	0.3g	0.3g	-
step 2D	0.3g	0.3g	-
JMA Kobe **3D 100%**	0.6g	0.82g	0.34g
step 2D	0.3g	0.3g	-
step 2D	0.3g	0.3g	-
Kashiwazaki R1 3D 50%	0.155g	0.34g	0.204g
step 2D	0.3g	0.3g	-
step 2D	0.3g	0.3g	-
JMA Kobe **3D 100%**	0.6g	0.82g	0.34g
step 2D	0.3g	0.3g	-
step 2D	0.3g	0.3g	-
Kashiwazaki R1 **3D 100%**	0.311g	0.68g	0.408g
step 2D	0.3g	0.3g	-

The testing series is listed in Table 3 and was reduced in comparison to the three-storey building, only one configuration was tested. The three chosen earthquakes, JMA Kobe, El Centro and Kashiwazaki, were firstly applied in 1D. The direction x as defined in Figure 3 corresponded to the North-South components acting along the long side of the building whereas direction y, the short side, was subjected to the East-West components. After the 1D quakes, the three earthquakes were applied with all three spatial components, in 3D and their original intensity. Analogously to the series on the three-storey building, a step input was applied between the earthquakes to observe the development of the eigenfrequencies and thus the damage. The percentages after the earthquake name and used spatial component in Table 3 indicate the scaling of the earthquakes, i.e. whether they have been applied at 100% intensity and their original PGA or if the PGA has been scaled.

Similar to the previous tests, also here no significant damage has been observed up to near-collapse, i.e. failure of one or more hold-downs. After the 3D tests, the bolts of the hold-downs had to be tightened and some nails in the shear angles close to building corners and openings had been withdrawn and needed to be re-inserted. Furthermore, when dismantling the building, no bending of the screws in the in-plane wall-to-wall joints could be observed.

Also the seven-storey building could withstand a whole series of large earthquakes although it had been designed with a behaviour factor of q = 3, reducing the seismic forces in comparison to a purely elastic behaviour. No permanent deformations could be observed, the building remained standing and kept its shape. The damaged connections could be repaired. Apart from embedment in the connections, the CLT elements were not damaged.

The measured deformation values for uplift and interstorey drift did not result in critical values in comparison to the preliminary test results on wall elements. For instance, the maximum uplift value on ground floor during the 3D JMA Kobe earthquake at 100% intensity resulted to 13 mm. This is a smaller value than the deformation at failure of 30 mm that was observed during the preliminary tests on the wall elements.

4. Final discussion

Both shaking table tests confirmed the earthquake resistance of CLT buildings. Neither the three-storey building nor the seven-storey building were seriously damaged. No permanent deformations could be observed after all tests although the buildings were subjected to a whole series of major earthquakes with peak ground accelerations of 0.5g and more. Furthermore, by means of the test results on the three-storey building, an indicative value for the behaviour factor q could be derived with whom earthquake engineers are able to carry out a simple and straightforward seismic design. With this indicative behaviour factor of q = 3, the seven-storey building was designed which could equally sustain a whole series of destructive earthquakes. With the test results on the seven-storey building and a numerical model, the indicative value of q = 3 could be confirmed. Other researchers also confirm the results and the generally good behaviour of CLT buildings under earthquake loading [11].

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More information on the SOFIE project: www.progettosofie.it
Information on the 3D shaking table: www.bosai.go.jp/hyogo/ehyogo/index.html