

# Experimental And Theoretical Assessment Of The Seismic Behaviour Of A Medium-Rise Hybrid Wood Construction System

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## SUMMARY:

The research work is aimed to identify the seismic behaviour of an innovative hybrid wood construction system (glulam frames and LVL panels) as extensively used within the Italian Government project for L'Aquila reconstruction after the 2009 earthquake (Progetto C.A.S.E.). Several wall assemblies were tested at National Research Council of Italy-IVALSA laboratories under quasi-static cyclic loading. Based on test results an entire full scale three storey building was designed to withstand the design maximum peak ground acceleration anticipated for Italian sites according to the recent Italian Rules for Buildings in seismic zones. At the same time a hysteretic model was set up to run under DRAIN 2DX program and calibrated on walls cyclic tests results. Finally the designed building was really erected and shaken on the EUCENTRE seismic table in Pavia (Italy) under a sequence of quakes in order to exploit the actual seismic performance of the building.

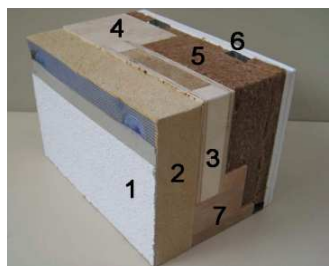
*Keywords: timber construction, shaking table, cyclic testing*

## 1. INTRODUCTION

The analyzed system is an innovative, timber frame system, which concentrates and summarizes several construction technologies in order to maximize its distinguishing features. This study is intended as a first step in determining the behavior against seismic actions.

## 2. DESCRIPTION OF THE CONSTRUCTION SYSTEM

The construction system object of this study, called Dolomiti Plus, is the result of years of experience and application. The structures of the main bearing framework are made of glued laminated timber beams and columns. The panels making up the outer and inner walls have no load-bearing function and thus allow a complete freedom in designing the spaces and volumes of the buildings. The wall panel has primarily a function of stiffening and stabilization of the entire frame against horizontal forces as well as of a vertical partition, being totally free from the static functions in relation to vertical forces which are instead held by the bearing framework in glued laminated timber beams and columns. The structure of the wall panel is based on some concepts of the "platform frame" system through the use of standard rectangular sections of small size and with a structural multi-layered fir panel (LVL) .



1. External plaster 8 mm
2. Wooden fiber 100 mm
3. Plus panel 60 mm
4. Main structure 160 mm
5. Wooden fiber 120 mm
6. Gypsum board 25 mm
7. Main structure 160 mm

**Figure 1.** External wall layers and main structures

The various multi-layered panels (floor, roof, wall) are designed for maximum thermal performance and acoustic insulation, according to legislation and according to the area in which you must undertake the construction. For example, the multi-layered panel wall of Figure 1 provides the values  $U$  [ $W/m^2 K$ ] which, coupled with good design, enable to create buildings with high energy concept, such as Class A+ buildings ( $<30 \text{ kWh/m}^2$  according to CasaClima standard - LEED), or even passive buildings.



**Figure 2.** Design model of the main structures; panel's frame on the production's bench and ready main wooden structures



**Figure 3.** Different assembling steps of a 3-storey Dolomiti Plus building in Italy located in L'Aquila and the building completed for -Progetto C.A.S.E.- (photo-credit ILLE Prefabbricati s.p.a)

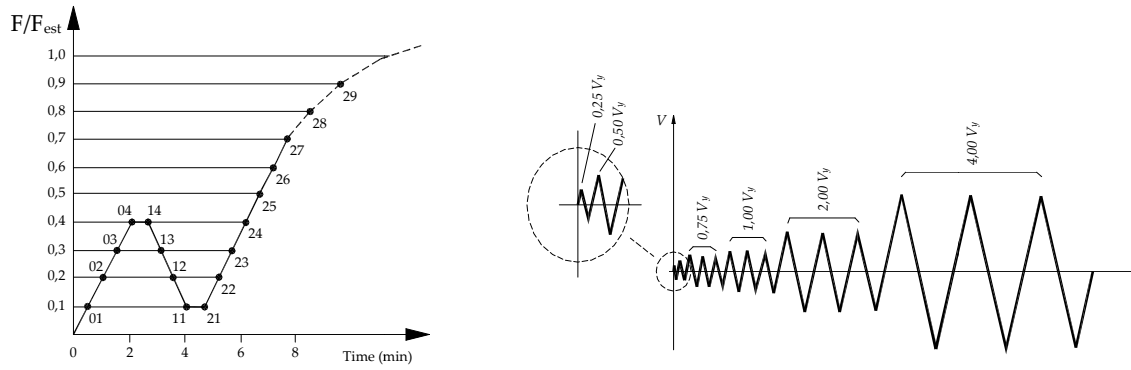
The wall panel is designed in parallel to the structures of the building, with all the holes for windows and doors and complete all work for proper implementation. The manufacturing process includes the workbench assembly of the elements of the framework by stapling and the LVL panel is fixed on the first side and hence the chosen insulating material is interposed. The assembly process takes place through gluing and pressing at controlled temperature and pressure. This process ensures the necessary rigidity and stability characteristics of the panel. The insulating materials chosen are of various types depending on the package that you wish. The bonding process requires the laying off of the adhesive on both sides of the work-frames structures and on both sides of the LVL panel. After this process it is possible to place the structures into the presses where they are left for a predetermined time and at a set temperature.

### 3. LABORATORY TESTS

In this chapter we analyze the details and the settings of the laboratory tests conducted on the wall produced by the company ILLE Prefabbricati Spa. We herewith describe the used types and standards going on to analyze the obtained data. We carried out two types of tests: a monotonic test with and without vertical load and then a cyclic test with vertical load.

#### 3.1. Standards And Specification Reference

The tests were carried out as proposed by the European standards with regard to wooden structures and to joints made with mechanical elements of connection (EN 26891:1993 - EN 12512:2003). We teste a prototype of the wall, a fact which appears to be a limitation of this procedure. In fact, to obtain statistically reliable results and data, you should perform a greater number of tests in order to give greater significance to the values obtained from the verifications. For the case we studied, average values were applied accepting a degree of approximation that still allowed safety conditions. The tests were carried out in accordance with the above mentioned standards and laws.



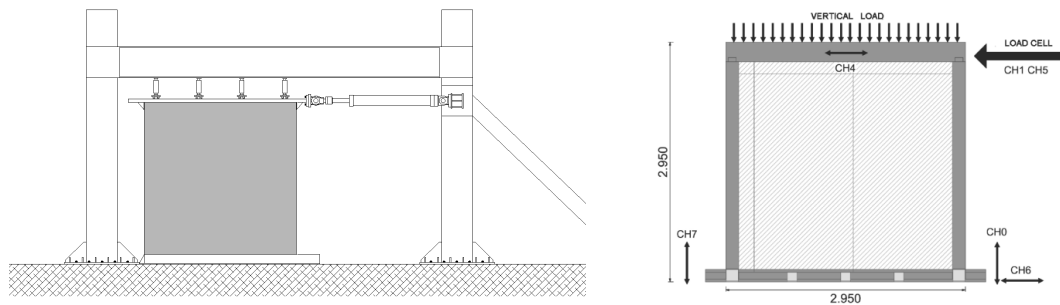
**Figure 4.** Monotonic test and cyclic test procedures

The cyclic test was carried out according to EN 12512 as shown in Figure 4.

### 3.2. Prototype Configuration And Tests Setting

#### 3.2.1. Bench test setting up

The tests were conducted at the laboratories of CNR-IVALSA Institute. The wall was rigidly fastened to the guides of the test bench by means of special anchors. The sizes of the wall are 2.95 m wide and 2.95 m in length.



**Figure 5.** Test bench layout and position of the loads - displacement controls

The test bench consists of a horizontal load cell and of a system of vertical load cells imposing the fixed load. The tests are performed in displacement control: a shift is imposed and simultaneously the applied force value is measured.

The various measurements of the displacements are registered and associated with the respective channel as pointed out in the scheme of Figure 5.

#### 3.2.2. Construction details of the wall prototype

The wall is made of a glued laminated timber portal which is tied to the panel called PLUSEPS60. The connections between the different elements are represented by screws. The vertical pillars and the main beam are fixed to the base platform by means of steel angles.



**Figure 6.** An example of ground connection and wood to wood connections

The wood-to-wood connections are made by means of mechanical juncture with screws having different diameters and different lengths. The connection between the upper beam and the pillars is represented by a mortise and tenon joint fixed with a screw HBS 10x400 tab washer. The curb-side consist of a larch wood beam. This element is shaped so as to create the right location to install the panel, that is tied to the structure by No. 28 screws of the type HBS 6x120 with a pitch of 40 cm inserted in inclined pre-drilled hole. The connections to the ground are steel angles obtained from normalized profiles with a thickness of 10 mm. In this case the junction to the basement is represented by a horizontal steel stringer fixed by means of a bolt M14, and in the case of concrete slabs, by means of expansion bolts. The two pillars are bound with angles clips of the type H150 connected with No. 9 screws HBS 6X120, while the basement is joint with 3 angles H120 and n ° 6 screws HBS 6x120.



**Figure 7.** Geometry of the prototype - connection to the ground (steel angles type H150 and H120) and Dolomiti Plus prototype wall on the test bench

For the production of the panel frame PlusEPS60 we use structural spruce wood C14 having a section of 55x35 mm. The spruce wood panel LVL has a thickness of 12.5 mm and a density of 600 kg/m<sup>3</sup> and the sheets size is 1.22 x 2.44m. The structures of the main frame are made of laminar wood of fir GL24c for pillars 16x16 cm and for the upper beam 16x 24 cm. The curb base is instead made with a three-blade larch cross section having a section of 12x12 cm.

### 3.3. Monotonic And Cyclic Laboratory Test

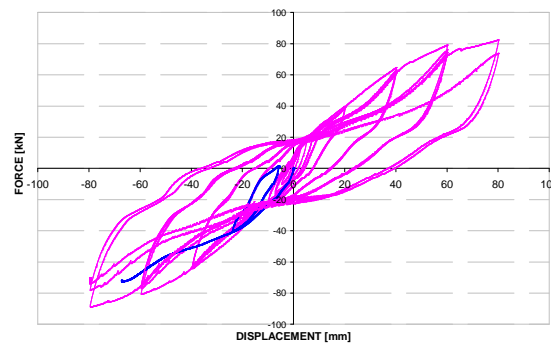
#### 3.3.1. Monotonic test

The data to be processed are collected from measurements carried out in several points of the prototype; the channel CH1 returns the values of the horizontal displacement of the load-cell while the channel CH5 returns the value relating to the measured force. The measurement of Channel CH 4 records the horizontal shifts due to the deformation of the beam. At the base of the prototype, the channels CH0 and CH7 measure the vertical shifts at both ends while the channel CH6 records the horizontal shifts near the end situated perpendicularly to the load cell. Initially a pretest was carried out by manual control and it was interrupted when a maximum load of 10 kN was attained: at this point the highest horizontal displacement of the load-cell was recorded as 7.38 mm. The second test was performed in absence of vertical load and was interrupted at the breakdown of No. 7 screws of the angle H150. This failure occurred near a load of 43.82 kN and with a displacement of 44.16 mm. It is possible to observe by the channel CH6, horizontal displacement equal to 3.88 mm near the peak force reached. In the first case a vertical lift at the base equal to 20.60 mm at the maximum load was registered. The most severely stressed point is the one situated perpendicularly under the point of application of the load-cell, in which the screws' breakdown occurred. The third test was conducted by placing a vertical load of 21.3 kN/m obtained by means of a series of load-cells set at a pressure of 80 bar. The inclusion of this load gives the system a condition of greater stability by increasing the effect due to the friction wood-to-wood and by counteracting the vertical lift. At the same time, it allows to simulate the presence of several levels imposed on the wall, and so reproducing the real situation of a building with its permanent and variable loads.

This test was stopped at reaching a value of deflection equal to 67.16 mm for a load of 72.74 kN. This type of procedure will cause slight discrepancies with the expected overlapping trend of the graphic of the cyclic test, but it provides results on the safety side.

### 3.3.2. Cyclic test

The cyclic test was conducted in displacement control that is by setting the shifts and recording the load obtained. After the first settlement cycles, we proceeded by increasing the strength and repeated three times the complete cycle before carrying on the test with new values of load. In this case, in order to continue the tests we decided to replace the plate H150 with a new plate H200, changing the position of the holes but still keeping the number and type of screws (No. 9 HBS 6x100). The other joints had no real breakdown or damage although it can be assumed that there were partial phenomena due to concentrated stress near the connections, so affecting the stiffness values of the system. The variations we introduced can explain the absence of overlapping in the graph of the monotonic test with the cyclic test graph. Generally, the first appears to be the envelope curve of the hysteresis cycle in the cyclic test. The maximum displacement achieved was 80.16 mm and the maximum force of 89.12 kN. This displacement was taken as the limit to fix the collapse criterion to be applied to simulations with the different accellerograms. The non-perfect symmetry of the curves represented in Figure 8 may be due to the fact that the reset elements absorb the load in a different way. Before stopping the test, No. 3 screws HBS 6x100 broke on the plate H150. Further failures occurred near the mortise tenon joint between the pillar and the upper beam, with a failure-cut on the external side of the wood. In the real situation the beam is continuous and therefore there is the same condition on both sides of the mortise, with the presence of an adequate quantity of materials. Other phenomena we observed concern the insertion of the heads of the screws fixing the panel to the structure and the failure of some screws. The locking screw of the mortise tenon joint was deformed: the failure occurred during the extraction phase. We also observed evidence of strong deformations due to concentrated stress



**Figure 8.** Cyclic test (pink line), estimated envelope curve (dot line), monotonic test (blue line)

## 4. PLASTIC NON-LINEAR MODELING

From the data obtained by the laboratory tests we get the parameters to perform the calibration of the model that will be used to simulate the seismic behavior of a single wall. The plastic nonlinear dynamic analysis involves the use of a specific software, the DRAIN2DX thanks to which we will afterward study and model also the behavior of a three-storey building with respect to several seismic events.

### 4.1.Drain2dx Code

The seismic behavior of wooden buildings mainly depends on the connections type and on the type of semi-rigid joints as well as on their plastic nonlinear behavior. The analysis software we applied - Drain2DX- exploits as inputs the time and rotation diagrams derived from cyclic tests on the wall prototype. The hysteresis cycle used to calibrate the model is shown in Figure 13 and represents a cycle with four slopes that fits with linear approximation, the results obtained from the cyclic test performed. To represent the behavior of the different joints it is necessary to calculate the different values of the slopes of the straight lines representing the values of different stiffness. The calculation is done by approximating the envelope curve and then, with a graphical method, by trial and error attempts to look for those values that, with appropriate adjustments, provide the best adaptation.

The values of stiffness  $K_1, K_2$  derive from  $M = \vartheta \cdot K$  where:  $M$ = moment,  $K$ =stiffness,  $\vartheta$  =rotation. The calibration of the model is obtained by iteration arranging the values of stiffness and controlling the overlap of the curves obtained with Drain2DX with those obtained from the cyclic laboratory test.

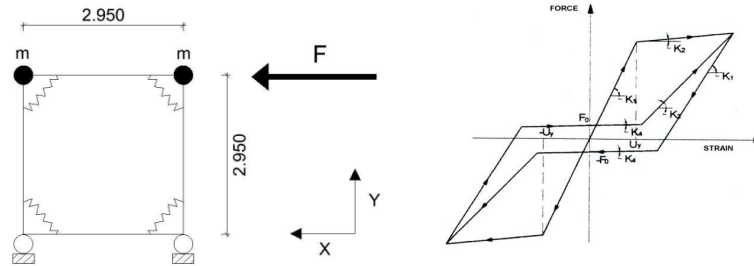
**Table 4.1.** Stiffness Values

$K_1$	$K_2$	$K_3$	$K_4$	$U_1$	$F_0$
7.1E+09	1.8E+09	2.8E+09	- 1	0.0028	1.3E+09

The degree of approximation is evaluated by calculating the energy dissipation. This value appears to be the integral of the curves of hysteresis in both the graph obtained from the laboratory test and in the graph obtained with DRAIN2DX.

#### 4.2. Description Of The Model

The type of wall studied is schematically represented as a wall frame consisting of two vertical elements (pillars) and two horizontal elements (beams) which are assumed to be infinitely rigid. Constraints are represented by rotational spring hinges that simulate the behavior of the whole wall. Since the displacements are infinitesimal, we approximately obtain the shifts in the direction of X axis, as a function of the applied force and of the rotation.



**Figure 9.** Model scheme and hysteresis cycle

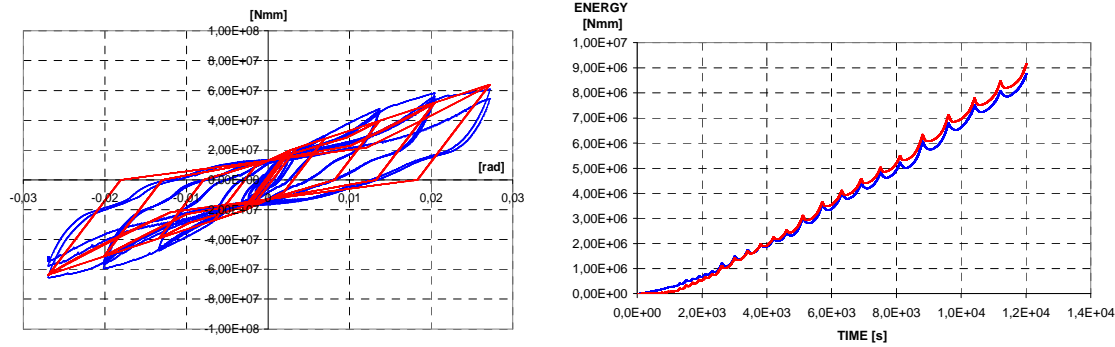
Simplifications are made at the level of symmetry and linear behavior, assuming that the springs (semirigid elements) are identical and that the force is distributed uniformly across the different nodes. According to the numerical values read on channels CH1 and CH5 of the test, all the displacements are transformed as a function of moment and rotation. The load of 21.3 kN / m was added to simulate the presence of the various levels and flats in the building and it is transformed into a mass value applied to the respective nodes and point of application of the seismic force. Finally, the structure is assumed as solidly anchored to the ground and therefore we neglected the displacements in the direction Y.

#### 4.3. Calibration And Results

After the calibration process and the parameters setting, we carried out the numerical processing that allowed us to optimize the mathematical model that represents the behavior of a single wall, in order to later apply it to the study of the whole building.

An initial comparison between the data obtained with the modeling and the test results concerns the parameters of displacement and the maximum force reached, indicating the calibration level of the model. From the analysis of the values summarized in Table 4.2. we notice that even at the level of rotation and moment, the values of the model confirm a high degree of adaptation to the data obtained with the laboratory tests. This will allow us to simulate the behavior of the wall subjected to the different seismic stresses.

Figure 10 represents the evolution of the value of dissipated energy. The total value of energy dissipation obtained from the test differs by 2.8% compared to the data obtained from the model. This difference reaches a maximum value of 6%.



**Figure 10.** Model (red line) compared to test results (blu line): force-displacement and energy dissipation

**Table 4.2.** Test And Model Maximum Displacement, Force, Rotation And Moment

Test				Model	
Displ.[mm]	Force[kN]	Rotation[rad]	Moment[Nmm]	Rotation[rad]	Moment[Nmm]
79.56	82.5	0.027	6.45E+07	0.026	6.35E+07

**Table 4.3.** Dissipated Energy Comparison

Dissipated Energy [Nmm]		
Cyclic test	Model Drain2DX	Range [%]
8064445.4	8295668.8	2.8

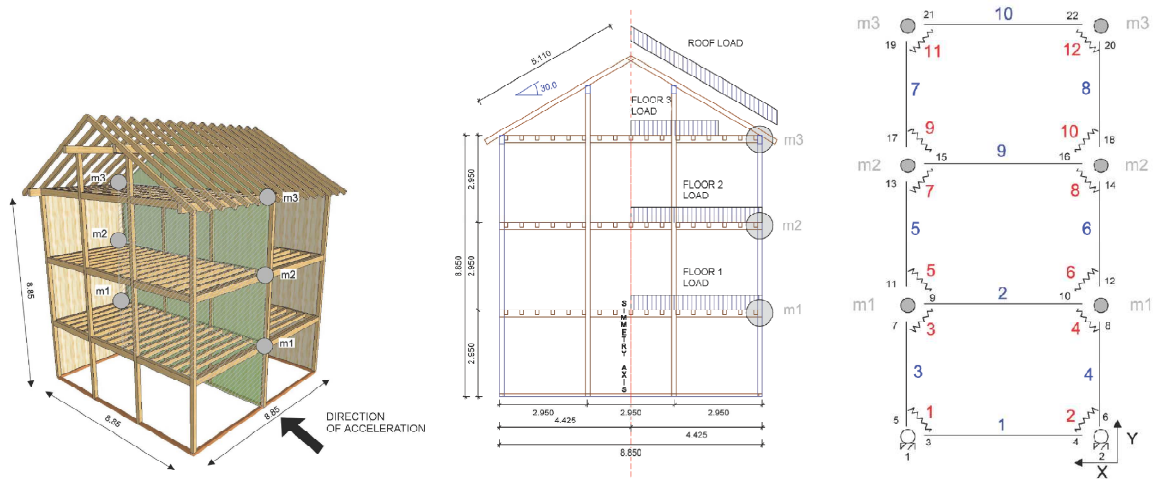
From the analysis of the values obtained from the test we find out the collapse criterion showing the achievement of a level of damage which is not recoverable under conditions of almost collapse. Generally the accepted value of maximum displacement is at  $0.8 F_{max}$  on the envelope curve subsequent to the achievement of displacement corresponding to  $F_{max}$ . On the base of the analysis of the test and to be on the safety side, we decided to indicate, as a collapse criterion, the displacement obtained near  $F_{max}$ . For the successive simulations, we consider actually that our structure is no longer able to bear external loads after reaching a maximum deformation of 80 mm. The first floor of the buildings erected with the framework system is the most susceptible to collapse and then this shift will concern the nodes near the first floor. Afterward we checked the displacements between the various floors, for which the same value limit is fixed.

## 5. SEISMIC SIMULATION

By means of the calibrated model we proceed to simulate the behavior and the response to the different seismic events by a multi-storey building erected with the system under study. Again we introduced the appropriate hypothesis in order to simplify the calculation but without affecting the safety degree and the reliability of the results.

### 5.1. Building Design And Model Setting

The building subjected to simulation consists of a basic constructive module defined by the model of the studied wall. In the plan the development of three modules of 2.95 m is projected while in height three floors of 2.95 m each. Furthermore we consider a roof angle of  $30^\circ$  with two roof-pitches.



**Figure 11.** Axonometric view of the wooden structures, and the position of the model wall (green)- loads and masses - Drain2DX model: semirigid elements (red), beam and columns (blue), masses (grey) and nodes (black)

The design is carried out according to Eurocodes. For calculation of the combined loads we consider  $\gamma = 1$  for permanent loads, while the variable loads are considered only for a fraction of 30%. The floors are considered to be rigid in the plane and moreover the building was simplified as having a perfect symmetry and a uniform distribution of the masses. It is possible to postulate that the stiffness of the wall obtained from the tests can be linearly extended to the stiffness of the walls to be modeled and that  $K1=3K$  where  $K1$  = stiffness of the whole wall of the building and  $K$  = wall stiffness  $2.95 \times 2.95$  m. The area of influence for the calculation of loads has the following sizes: 4.425 m by 8.850 m. On the floors in addition to the structures self weight, the weight of the outside wall elements was added.

$$F_d = G_k + \sum \psi_{2i} Q_{ki} = G_k + 0.3 Q_k \quad (5.1.)$$

where  $G_k$ = permanent loads (roof, floor, walls)  $Q_k$ = variable loads ( $q_k = 2.0 \text{ kN/m}^2$ )

From (5.1) we obtain the loads acting on the structure which are then transformed into masses to represent the action of the seismic force, as summarized in Table 5.1.

**Table 5.1.** Values Of The Total Loads And Masses Included In Drain2dx

	Permanent	Variable	Total	Masses
Total loads	[kN]	[kN]	[kN]	[Ns <sup>2</sup> /mm]
Roof / ext wall	65.70	-	-	-
Floor 3rd	109.59	23.5	198.79	m3+m3 20.38
Floor 2nd	109.59	23.5	200.88	m2+m2 20.50
Floor 1st	109.59	23.5	200.88	m1+m1 20.50

Figure 10 describes the geometric schema use by FEM model, as the result of a bi-dimensional analysis providing the displacements for all nodes in each instant of the time (time step  $t=0.01$ s). The simulations are carried out using the accelerograms of several earthquakes, and scaling their intensity according to the values limits as specified by the standard for the Italian national territory.

## 6. RESULTS

### 6.1. Simulations Results

The results obtained from the elaboration provide the displacement values for each node according to the scheme in Figure 10. For peak acceleration levels equal to 0.35g the building is able to absorb the deformations on the first floor for all earthquakes. Only for the Kobe earthquake we obtained values which are slightly beyond the permissible limit of 8.0 cm. With reference to the earthquakes that took place in Italy, on the contrary, the recorded deformations are quite below the allowed limit. This

outcome was also on the safety side because we examined an element of two-dimensional wall that is not subjected to the contribution of the entire three-dimensional structure. Referring to the level of seismicity of the Italian territory and taking into account a maximum value of  $PGA_{u,code}=0.35g$ , we put the building in extreme conditions.

**Table 6.1.** First Floor Maximum Displacement With  $PGA_{u,code}=0.35g$

Earthquake	Max. displacement [cm]
Brienza 23/11/1980 (Italy)	2.00
El Centro 19/05/1940 (USA)	3.94
Izmit 17/08/1999 (Turkey)	4.89
Kobe 16/01/1995 (Japan)	8.68
Nocera Umbra 26/09/1997 (Italy)	1.04
Tolmezzo 6/5/1976 (Italy)	1.70
L'Aquila 6/04/2009 (Italy)	1.79

The collapse of the wooden framework buildings in fact are caused in most cases by the maximum displacements at the first floor, on which the rest of the structure then falls down. As for the relative displacements between the floors we considered the values obtained at the same instant of time. Even in this case the limit of collapse of 8 cm was reached only in the situation of the Kobe earthquake scaled to 0.35 g. For all other earthquakes scaled to 0.35 g the results were below that value. For 0.25 g and for 0.15 g the safety level appears to be much higher, with values of relative displacements between the floors much lower than the limit value.

## 6.2. Calculation of the structure factor q

Eurocode 8 defines the structure factor q, which allows to reduce the forces derived from the linear analysis considering the non-linear response, according to the different materials, the structural system and the type of design. After modeling the behavior of the building thanks to the laboratory tests and after choosing the collapse criterion, the q factor is derived from the relation between the peak acceleration  $PGA_{u,eff}$ , that causes the collapse of the building and the value of the peak acceleration  $PGA_{u,code}$ , determined by the specifications.

**Table 6.2.**  $PGA_{u,eff}$  And Calculated q Factor

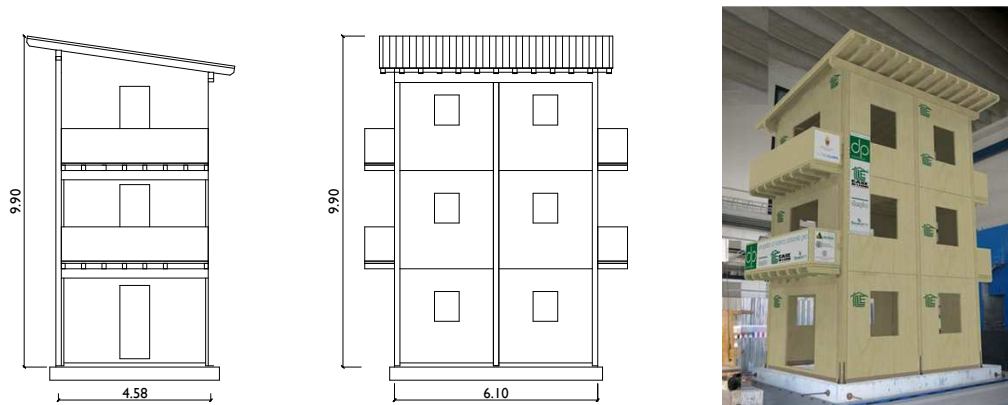
Earthquake	$PGA_{u,eff}(g)$	Calculated q factor
Brienza (Italy)	1.54	4.40
Nocera Umbra (Italy)	2.69	7.69
Tolmezzo (Italy)	1.41	4.03
L'Aquila (Italy)	1.45	4.14

In relation to the adopted collapse criterion (inter-floor maximum displacement equal to 8 cm), the results listed in Table 6.2 show that for all the studied Italian earthquakes and for the analyzed building prototype we obtain a q factor of structure higher than 4.

## 7. SHAKING TABLE TESTS

The presented tests and numerical calculations allowed to develop a 3-storey building prototype shaken on the EUCENTRE seismic table in Pavia (Italy) under a sequence of quakes in order to exploit the actual seismic performance of the building. The structure was built with the system described in the previous paragraphs. In order to achieve more data about the cyclical behaviour of the wall system used to create the prototype, 4 further cyclical tests were carried out at IVALSA laboratory. The experimental data obtained from cyclical tests and a 3D FEM model has allowed to design the 3 storey building tested on shaking table which dimension are the following: 6,10 m by 4,7 m, the height is equal to 9,9 m. The floors have been loaded by concrete elements in order to simulate the vertical load described by (5.1.). The design permanent loads were equal to  $1.9 \text{ kN/m}^2$  and the

variable loads equal to  $q_k = 2.0 \text{ kN/m}^2$ . The prototype has been designed according to Eurocode 5 and Eurocode 8 considering a PGA equal to 0.7g.



**Figure 12.** Shacking table test, the prototype

The building has been tested under a sequence of quakes, Table 7.1, the last quake has a maximum acceleration (PGA) of 1.2 g. The chosen accelerogram is the Montenegro quake recorded in Hercec Novi. At the conclusion of the series of shocks, the building did not show evident failure mechanisms.

**Table 7.1.** Sequence Of Quakes

Date	Accelerogram	PGAu,eff(g)
16-05-2012	Montenegro	0.25
18-05-2012	Montenegro	0.5
19-05-2012	Montenegro	1.00
19-05-2012	Montenegro	1.20

## 8 RESULTS

The analysis carried out require further tests using different elements of the wall, different prototypes of buildings in scale and a larger number of accelerograms, but still provide promising indications. On the base of the results obtained, it can argue that the construction system in timber framework called Dolomites Plus, if properly designed, presents an excellent behavior against seismic actions. As for the value of the structure  $q$  factor, the presented analysis show that this construction system can be considered as highly dissipative. The author's opinion is that the presented building system deserves to be more deeply examined. There is justification for the use of a  $q$  structure factor not lower than 3 .

## AKCNOWLEDGEMENT

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