Calibration of the numerical model of a timber structure

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SUMMARY:
The work presented here focuses on the calibration of a numerical model of a 4 storeys timber panel building. To carry out the study, after the execution of dynamic tests on a full-scale four-storeys structure, numerical and experimental results have been compared both in terms of dynamic characteristics and seismic response. Initially, the considered specimen and the results obtained from shaking table tests will be presented with particular attention to the modal characteristics, obtained through dynamic identification, and to the maximum storey-shear and storey-displacement evaluated from dynamic tests. Then, the adopted linear elastic finite element model will be described with particular attention to the calibration process used for the distribution of the Young's modulus of the material representing the panels. Finally, the two sets of results (experimental and numerical) will be compared in terms of storey-shear and storey-displacement to verify the accuracy of the macro-element model.

Keywords: timber structure, shake table test, model calibration

1. INTRODUCTION

The earthquake occurred in L'Aquila (Italy) on April 6th, 2009 showed the vulnerability of buildings realized with traditional techniques, such as masonry or reinforced concrete. After that event, a number of experimental campaigns were carried out at Eucentre (European Centre for Training and Research in Earthquake Engineering, Pavia, Italy) aiming to the investigation of the seismic behaviour and to the performance evaluation of alternative construction technologies. A number of interesting possibilities such as concrete sandwich panels, Insulating Concrete Forms (ICFs) panels, timber panels structures have been considered in the last years.

Timber structures are known to be very efficient in terms of energy performance. On this point the Italian regulations, implementing the one approved by the European Union, has set very strict rules on the energy performance of new buildings in order to reduce energy dependence on countries outside the European Union. This factor is leading to a wide rediscovery and use of timber structures since the energy efficiency of a system built with traditional techniques can be reached using thinner elements. Moreover, the costs, given by the industrialization of structural panels, are reaching a competitive level compared to traditional housing systems.

Energy savings, cost savings and construction speed are certainly key points for the competitiveness of this construction technique. Although a number of studies can be found in literature, regarding both the design (*e.g.* Boding *et al*. [1982], Modena *et al*. [2005]) and the seismic behaviour (*e.g.* Ceccotti *et al*. [2007]) of wooden structure, the seismic response can however be further investigated. An extensive experimental campaign, ending with the shake table testing of a full scale four storeys building, and a numerical study have been carried out. Comparison has been performed between the experimental evidence and the numerical results obtained using a finite element (FE) model implemented in ProSA, a modeller developed at Eucentre. The software models each structural element with an equivalent frame macro-element, as typical for masonry structures, and performs linear, modal and response spectrum analyses. The distribution of the elastic modulus of the elements of the FE model has been calibrated using both the storey shear-displacement behaviour recorded during the dynamic tests and the results of the dynamic identification tests.

In order to evaluate the accuracy of the adopted modelling assumptions, the results of the modal response spectrum analysis have been compared in terms of storey-shears and displacement profile to the outcomes of the tests.

2. DESCRIPTION OF THE SPECIMEN AND INSTRUMENTATION SET-UP

A four-storeys wooden panels structure has been tested on the uni-axial shake table of Eucentre TREES Lab (Laboratory for Training and Research in Earthquake Engineering and Seismology). The complete structure was 6.94 m by 5.30 m in plan and was 11.55 m tall. The building was realised assembling prefabricated timber panels realised with a wooden frame covered with masonite hardboards made of steam-cooked and pressure-moulded wood fibres. The total thicknesses of the external and internal panels were respectively 192 mm and 152 mm. The following Fig. 2.1 shows some of the construction phases.

Figure 2.1. Construction phases of the tested structure

The connections between the different parts of the buildings (*i.e.* panel-panel, panel-slab, panel-steel foundation) have been mainly realised with three types of steel (S275) connectors: brackets, hold-down and tie-down. More in detail, the brackets are squat steel elements assuring continuity between the vertical and the horizontal elements and transferring shear forces, Fig. 2.2.(a) shows some bracket installed on the steel foundation. Fig. 2.2.(b) shows some hold-down, the longer steel element in depicted in the figure, connecting the panels to the foundation and transferring the vertical tensile stresses in order to avoid the rocking of the panels. The vertical continuity between the panels at different storeys was realised with the tie-down connectors, some are shown in Fig. 2.2.(c): the role of this type connector is to transfer vertical stresses due to the in-plane bending moments. In addition to these connectors, several steel screws and bolts have been installed too: screws have been used to connect panels and slabs, while bolts have been used for the panel to panel connections (see holes close to the building corner in Fig. 2.2.(c)).

(a) (b) (c)

Figure 2.2 Steel connectors: (a) brackets; (b) hold-down; (c) tie-down

During the dynamic tests the structural behaviour of the specimen was monitored using more than 100 instruments installed on the building. For what concern the issues treated in following paragraphs, the most interesting quantities to be monitored were (*i*) the acceleration at each floor and (*ii*) the displacement along the height of the building. The first was monitored using both capacitive and piezotronic accelerometers, six accelerometers were installed at each floor to measure longitudinal and transversal acceleration at three location across the slab. A machine vision system, the details of which can be found in Lunghi *et al* [2012], was used to measure the displacements of several markers applied to the façade of the building. Besides these instruments, the monitoring was completed using several displacement transducers to record the shear distortions of the panels and possible relative sliding movements of the panels and between the panels and the foundation. Additionally, some strain-gauges were installed on the hold-down connectors to monitor their stress.

3. DYNAMIC TESTS

Multiple dynamic tests at increasing amplitude were performed at Eucentre TREES Lab. Initially, the 2009 L'Aquila earthquake was used as seismic input scaling its record at different amplitudes, the peak ground acceleration varied from 0.33 g to 1.30 g with the steps reported in the following Table 3.1. At the end of the testing campaign the 1995 Kobe earthquake (PGA equal to 0.8 g) was reproduced. Table 3.1 also summarises the maximum base shear recorded during the tests. It is interesting to note that, comparing the data relative to the L'Aquila earthquake record, the relation between the PGA and the base shear is almost linear, meaning that the structure was still in the linear range and did not yield. This was in agreement with the results of the performed visual inspections from which was not possible to find any meaningful damage despite the high PGA reached during the tests.

Table 3.1 Base shear determined from each dynamic test carried out

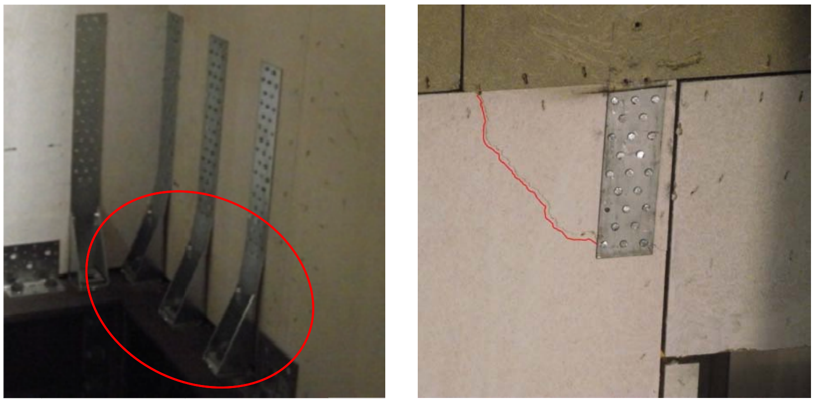
Record	L'Aquila						Kobe
PGA [g]	0.33	0.66	0.90	0.90 (B)	1.10	1.30	0.80
Base shear [kN]	202	352	465	473	523	631	564

The changes of the dynamic characteristics of the structure were monitored during the testing campaign performing dynamic identification after each shaking. Table 3.2 partially shows the results of the dynamic identification reporting the first two natural frequencies, respectively first transversal and first longitudinal mode. From these values it is possible to note that the variation of the two main frequencies was quite small confirming the very low level of seismic induced damages. For sake of completeness, it must be pointed out that the frequency increase found after the first 0.90 g test was due to a tightening of the bolted connections between the panels.

Table 3.2. First two periods of the building found with dynamic identification

	Pre-test	Post 0.33g	Post 0.66g	Post 0.9g	Post 0.9gB	Post 1.1g	Post 1.3g	Post Kobe
f1 [Hz]	3.94	3.69	3.62	3.63	3.81	3.81	3.56	3.25
f2 [Hz]	4.56	4.37	4.12	4.06	4.25	4.06	3.87	3.56

Generally speaking, the seismic behaviour of the tested wooden structure was satisfactory. The multiple shaking used to excite the structure was not able to induce any significant damage. The only visible damages were found on some of the hold-down connectors, that yielded in tension and buckled returning at rest, see Fig. 3.1.(a). Besides, some small cracks were found on the masonite hardboards close to the corners of the openings were the tie-down connectors were installed, see Fig. 3.1.(b).



(a) (b)

Figure 3.1 Seismic induced damages: (a) buckled hold-down; (b) crack on the panel hardboard

Finally, it is worth mentioning that from the envelops of the shear-displacement hysteretic cycles at storey level (see Fig. 4.2) it was possible to estimate the equivalent viscous damping ratios: the average value for the whole structure equalled about 20%.

4. NUMERICAL MODELLING OF THE SPECIMEN

The finite element (FE) model of the structure was realised using Prosa, a modeller developed at Eucentre. Using this software, the user can create a 3D model of the structure using macro-elements that represent the structural elements. Each of them, in this case panels, spandrels and riddles, is then automatically converted in an elastic macro-frame element adopting a modelling strategy commonly used for the the masonry structures. The software also creates a number of ficticious “rigid” elements that connects the elastic frames representing the structural elements. Additionally, the software allows the modelling of the floor slabs used to distribute the gravity loads between the vertical elements and, if desired, to define rigid diaphragm costraints. Further discussion about the software and its modeling solution can be found in Peloso *et al* [2010]. The following Fig. 4.1 shows the 3D model of the structure, on the right, and the corresponding FE model, on the left.

It has to be noted that the different colors of the elements (see right side of Fig. 4.1) correpond to the different adopted elastic modulus. The determination of such values were done on the basis of an optimisation process taking advange of the results of several experimental studies. Prior to the execution of the shake table tests, the FE model was used to roughly estimate the maximum expected force level in order to define the test set-up. In this phase, the stiffness of the elements representing the panels was estimated using the results of a previous experimental campaing performed at Eucentre involving pseudo-static cyclic tests on single panels and structural sub-assemblies.

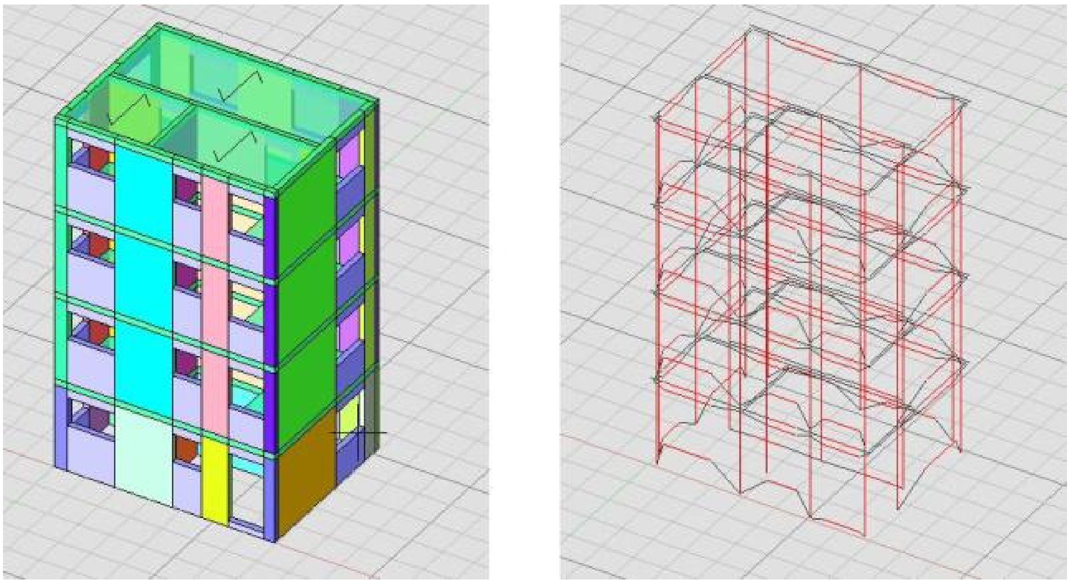


Figure 4.1 Equivalent macro-frame modelling of the building

After the execution of the dynamic tests, the model was recalibrated optimising the stiffness distribution in order to fit the results of the dynamic identifications, both in terms of natural periods and modal shapes, and the storey hysteretic behaviour showed during the shake table tests, in terms of equivalent secant stiffness. As an example, Fig. 4.2 shows the hysteretic cycles of the second storey of the building, derived from the processing of the recorded data relative to the 0.9 g test. The graphics also shows the equivalent secant stiffness that has been used for the optimisation process.

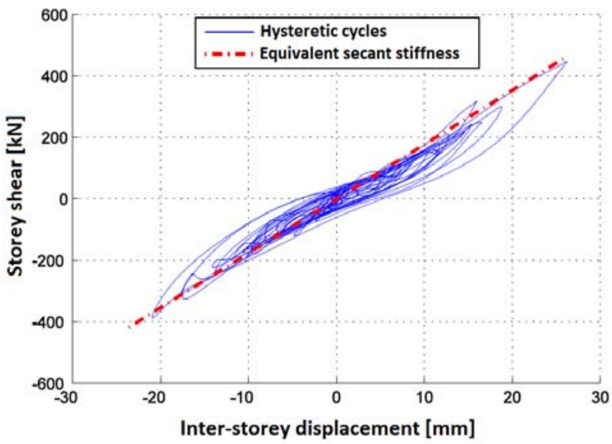


Figure 4.2 Example of storey shear – inter-storey displacement hysteretic cycles

5. COMPARISON BETWEEN EXPERIMENTAL AND NUMERICAL RESULTS

The FE model has then been used to estimate some of the response quantities characterising the behaviour of the timber structure under three of the dynamic excitations simulated during the testing campaign: in particular the tests with PGA equal to 0.33 g, 0.66 g and 0.90 g have been considered. It has to be underlined that, since the dynamic characteristics of the structure were varying test after test, the FE model calibration has been repeated in order to update the stiffness distribution to each of the analysed cases.

The previously described model has been used to perform Modal Response Spectrum Analysis (MRSA): the seismic input was represented by the actual acceleration response spectrum as derived from the feed-back signal acquired by the control system of the shake table. Clearly, according to the hysteretic behaviour of the building, a 20% equivalent viscous damping has been used for the evaluation of the acceleration spectrum. The obtained results have been compared in terms of maximum storey displacement and maximum storey shear: clearly, as the MRSA only gives the maximum expected values for each parameters, these have been compared to the maximum absolute values obtained from the tests.

The experimental maximum storey displacement have been obtained directly from the machine vision system used for the monitoring of the displacements, while the values of the shear have been obtained from the signal acquired by the accelerometers installed on the specimen. The three acceleration records acquired at each floor along the direction of motion (*e.g.* long side of the model as depicted in Fig. 4.1) have been averaged and multiplied by the relative value of mass to obtain the inertia force at each storey. Clearly, a checks have been performed in order to verify that the global equilibrium of the system (building and shake table) was respected.

The following Fig. 5.1, 5.2 and 5.3 show, on the left side, the envelops of the displacement profiles at the considered test intensities. The shape of the displacement profile was fitting quite well the experimental results but the estimated displacements were slightly underestimating the test outcomes. Nevertheless the results should probably be considered to be quite satisfactory as such difference could be partially due to the precision of the adopted analysis method. As expected, the average error is increasing with the level of the shaking, *i.e.* with the non-linearities of the real model that cannot be accounted by the FE model nor by the MRSA method. On the right side of Fig. 5.1, 5.2 and 5.3, the storey shear distributions are showed: as for the displacements, the experimental values are slightly exceeding those estimated by the numerical analysis.

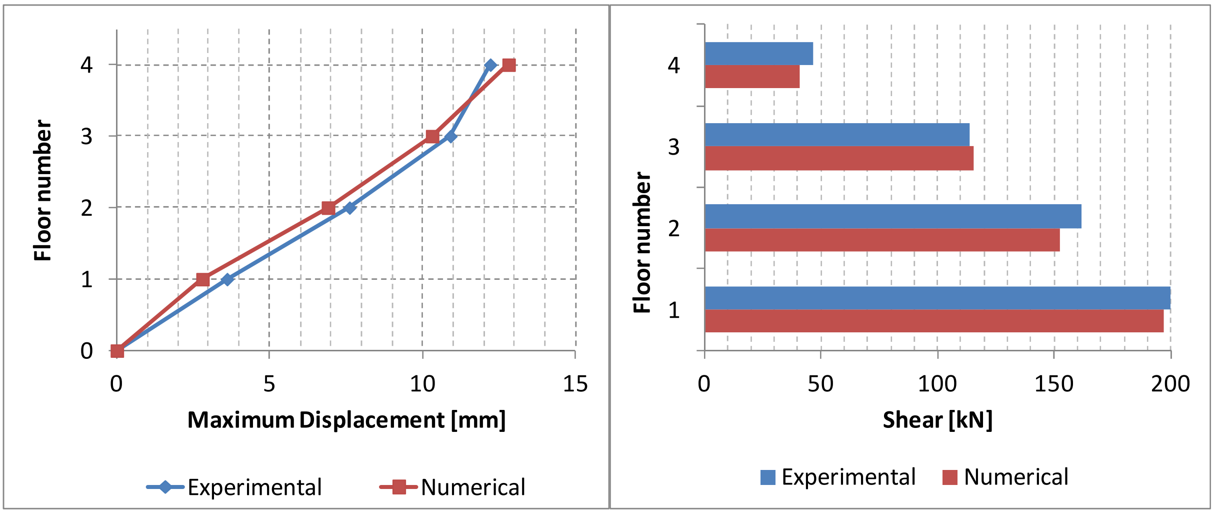


Figure 5.1 Test 0.33g: storey displacements (left) and shear values (right)

Table 5.1. Test 0.33 g: summary of the results

Storey	Displacement			Shear		
	Experimental	Numerical	Error	Experimental	Numerical	Error
#	[mm]	[mm]	[%]	[kN]	[kN]	[%]
1	3.6	2.8	-22.2	200.1	196.9	-1.6
2	7.6	6.9	-9.2	161.9	152.5	-5.8
3	10.9	10.3	-5.5	113.7	115.5	1.6
4	12.2	12.8	4.9	46.9	41.0	-12.6
		Average error	-8.0		Average error	-4.6

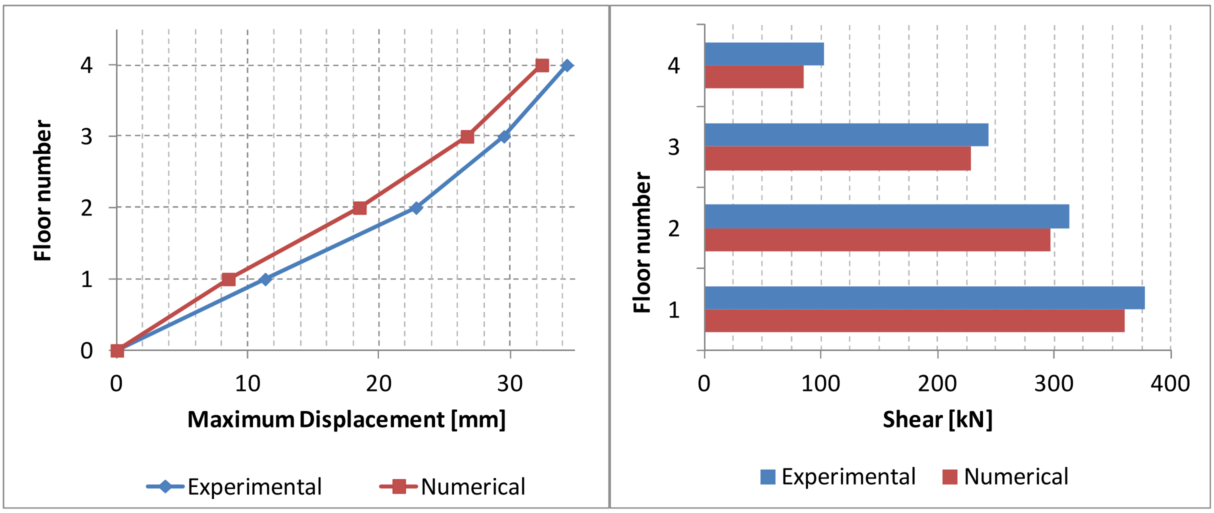


Figure 5.2 Test 0.66g: storey displacements (left) and shear values (right)

Table 5.2. Test 0.66 g: summary of the results

Storey	Displacement			Shear		
	Experimental	Numerical	Error	Experimental	Numerical	Error
#	[mm]	[mm]	[%]	[kN]	[kN]	[%]
1	11.3	8.5	-24.8	377.3	360.6	-4.4
2	22.8	18.5	-18.9	313.6	296.6	-5.4
3	29.5	26.7	-9.5	244.3	228.7	-6.4
4	34.3	32.4	-5.5	103.0	85.1	-17.4
		Average error	-11.0		Average error	-9.7

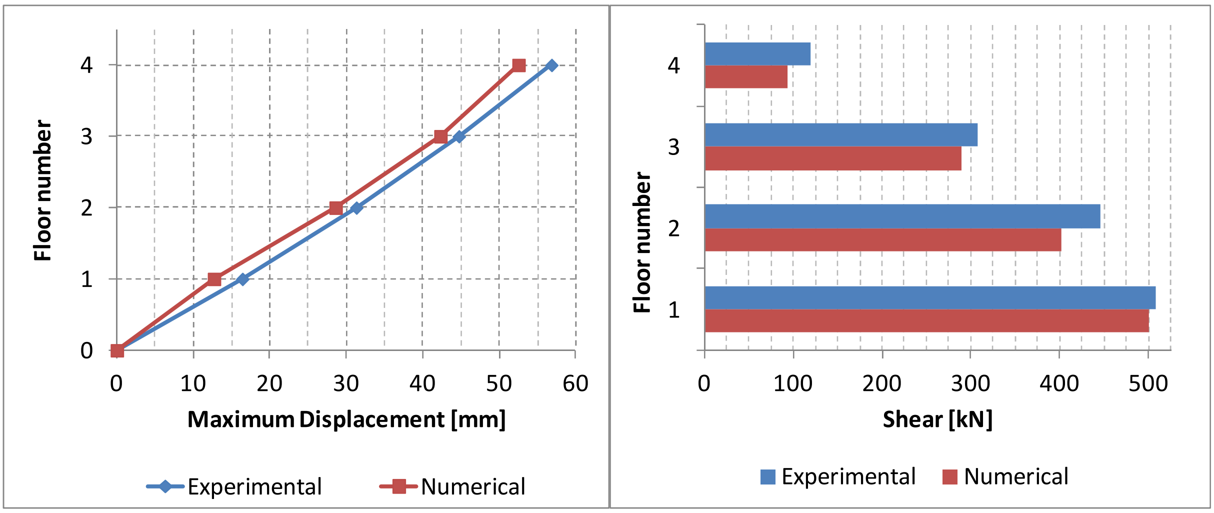


Figure 5.3 Test 0.90g: storey displacements (left) and shear values (right)

Table 5.3. Test 0.90 g: summary of the results

Storey	Displacement			Shear		
	Experimental	Numerical	Error	Experimental	Numerical	Error
#	[mm]	[mm]	[%]	[kN]	[kN]	[%]
1	16.4	12.7	-22.6	508.6	500.3	-1.6
2	31.3	28.6	-8.6	446.6	401.4	-10.1
3	44.7	42.3	-5.4	308.2	289.6	-6.0
4	56.8	52.5	-7.6	119.1	94.0	-21.0
		Average error	-14.7		Average error	-8.4

Tables 5.1, 5.2 and 5.3 shows the numeric quantities depicted in previously discussed graphics: besides the displacement and shear values, the errors affecting the estimated quantities are reported. On the displacement side, it is interesting to note that the average difference between the numerical estimate and the experimental results ranges between 8% and 15% of the correct value. Lower average errors affect the estimated shear values being always not higher than 10%.

Besides the limitations characterising the adopted analysis method, a refinement of the results could probably be achieved modifying the equivalent viscous damping chosen for the structure. Since both displacement and shear values were under-estimating the experimental ones, there is a chance of having better results adopting a reduced global equivalent viscous damping: further investigations on this aspect are currently going on.

6. CONCLUSIONS

After the 2009 L'Aquila earthquake, that once more highlighted some limitations of the Italian building stock, and taking advantage of new European and Italian regulations that enforces the respect of stricter limits on the side of the energy-efficiency, a number of building techniques, alternative to the traditional reinforced concrete and masonry, are becoming more popular. Between these techniques, timber panel structures are surely acquiring importance in the Italian context. Nevertheless, their seismic behaviour still need to be further investigated in order to assure a proper response in case of seismic excitation. Furthermore, there is a need for the definition of a numerical modelling strategy to be used for the design of such structures.

In this framework, an experimental and numerical campaign took place at Eucentre (Pavia, Italy) ending with the dynamic testing of a four-storey full scale timber building. The paper briefly reviews this research work starting with the description of the specimen used for the shake table tests and discussing the instrumentation set-up. A modelling solution is then introduced: the timber panel building is modelled with equivalent linear macro-frame element, as usually done for masonry structures. The main positive aspect of the adopted modelling strategy is the very limited computational effort since the model directly derives the internal actions on the structural panels. Furthermore, the creation of the FE model of the panel building is quite simple since the proposed modelling solution has been implemented in ProSA, a software developed at Eucentre.

The focus is then shifted to the calibration of the FE model: the stiffness, *i.e.* the elastic modulus, of the equivalent linear frames have been tuned to fit the results of some tests performed on the real structure. In particular, the stiffness of the elements have be calibrated in order to fit both (*i*) the storey shear-displacement behaviour showed during the dynamic tests and (*ii*) the eigen-quantities, *e.g.* modal shapes and natural periods, determined through dynamic identification tests.

After the calibration process, repeated for each of the analysed test intensities, the FE model have been used to perform Modal Response Spectrum Analysis (MRSA). The obtained results have been compared in terms of maximum storey displacement and shear with the outcomes of the experimental tests. A general good agreement has been found for both the considered quantities: the numerical displacement and shear profile fitted quite well the experimental evidence. A slight underestimation of the response quantities have been observed but the average error was always below the 15%. This behaviour of the numerical model could be due to the limitation of the MRSA method, that is probably not perfect for this case since the maximum considered PGA goes beyond 1.0 g and some non-linear behaviour is present, and to a possible over-estimation of the equivalent viscous damping.

Finally, it could be stated that the experimental tests performed on the timber panel building were satisfactory, since the building was able to sustain multiple shaking reaching a maximum PGA level equal to 1.3 g (obtained scaling the record of the 2009 L'Aquila earthquake). At the end of the testing campaign there were no significant damages nor loss of horizontal and vertical load-carrying capacity. The dynamic identification tests showed a limited modification to the dynamic characteristics of the building, proving once more that only a moderate reduction of the lateral stiffness of the building took place. Additionally, the quite accurate results of the MRSA performed using the proposed FE modelling solution indicated that such strategy can be successfully adopted for the estimation of the internal actions induced by a seismic motion. Further improvement can be achieved, and are currently under investigation, refining the global equivalent viscous damping that was probably slightly overestimated.

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