

Shaking table testing of a multi-storey post-tensioned timber building

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2014 NZSEE
Conference

ABSTRACT: This paper describes results of shaking table testing of a post-tensioned timber frame building in the structural laboratory of the University of Basilicata in Potenza, Italy. This experimental campaign is part of a series of experimental tests in collaboration with the University of Canterbury in Christchurch, New Zealand. The specimen was 3-dimensional, 3-storey, 2/3rd scale and constructed using post-tensioned timber frames in both directions. The structure was tested with and without dissipative steel angle reinforcing which was designed to yield at a certain level of drift. These steel angles release energy through hysteresis during seismic loading, thus increasing damping. Testing was performed up to a maximum PGA of 0.77g with and 0.58g without the dissipative reinforcing. At comparable levels of PGA the addition of the reinforcing reduced drifts by 32% without increases in peak floor accelerations. Test results were also compared favourable against numerical blind predictions using the RUAUMOKO 2D and SAP2000 structural analysis programs.

1 INTRODUCTION

This paper describes results of shaking table testing performed on a multi-storey post-tensioned timber frame building in the structural laboratory of the University of Basilicata (UNIBAS), Potenza, Italy. This work is part of a collaborative experimental campaign between UNIBAS and the University of Canterbury (UoC), Christchurch, New Zealand. The full study will evaluate the feasibility of applying jointed ductile post-tensioning technology, originally conceived for use in concrete structures (Priestley et al. 1999) and successfully adapted using large laminated veneer lumber members, to glue laminated timber (glulam) incorporating new methods of dissipative reinforcing. The project will evaluate the seismic performance of the system and further develop it for use in multi-storey timber buildings. The post-tensioned timber concept (under the name PRES-LAM) has been developed at the University of Canterbury and extensively tested in the structural laboratory of the university (Newcombe et al. 2010; Palermo et al. 2005; Smith et al. 2007). This technology enables the design of buildings having large bay lengths (8-12m), reduced structural sections and lower foundation loads with respect to traditional construction methods.

The PRES-LAM concept uses post-tensioning technology in order to connect structural timber elements. This post-tensioning is normally combined with dissipative reinforcing devices (a 'hybrid' structure). The structural response of a post-tensioned timber frame centres on the moment-rotation response of its connections which is a combination of the post-tensioning moment capacity (M_{pt}) and moment capacity provided by the reinforcing (M_s). During design, an engineer can select what percentage of the moment demand to allocate to the post-tensioning with the ratio between M_{pt} and the total moment capacity ($M_t = M_{pt} + M_s$) giving the factor β (Figure 1).

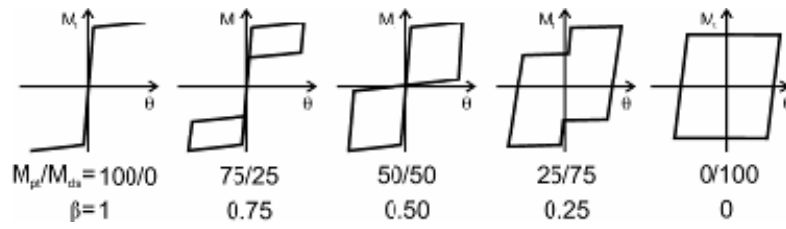


Figure 1. Moment response with varying levels of the parameter β .

Although a simple concept, this ratio provides the cornerstone in the understanding of system performance. Clearly, during design this choice affects both damping and moment capacity of the system and therefore changing this value will have a direct effect on both capacity and demand.

In the current stage of the project a 3-dimensional, 3-storey timber structure (Fig. 2) has been dynamically tested in real time in the UNIBAS lab. During the experimental campaign the size of the structural members, building layout and mass was not altered, however the frame was tested with and without the use of dissipative reinforcing. The dissipative devices used were based on yielding steel angles which activate at low drift levels, both increasing the moment capacity of the system and adding energy dissipation (thus reducing seismic load through damping) without inducing plastic deformations in other elements. This paper will describe briefly the detailing and testing set-up of the experimental model. Following this, testing results with and without dissipation will be presented and studied in order to evaluate the impact that the design choices have on the frame dynamic response. Finally comparisons between experimental outcomes and SAP2000 and RUAUMOKO numerical predictions will be shown.

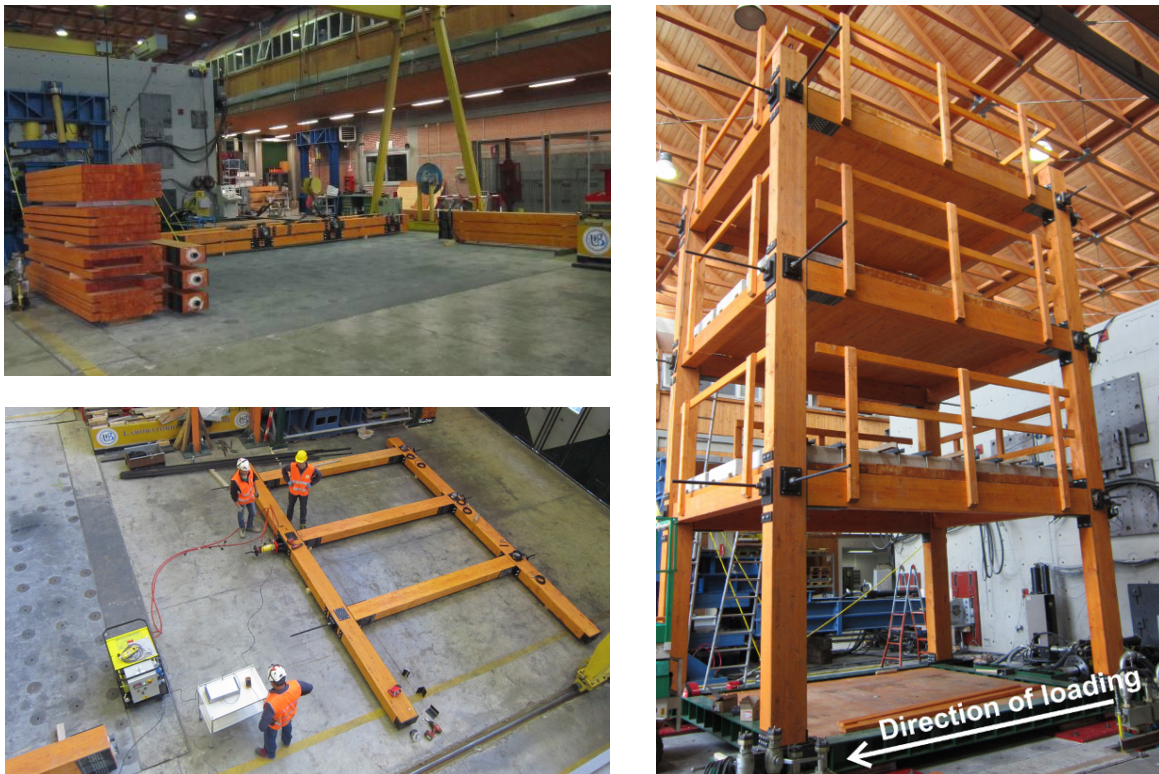


Figure 2. Experimental model constructed in UNIBAS lab.

2 TESTING STRUCTURE

The building has been designed to represent an office structure (live loading $Q = 3 \text{ kPa}$) with the final floor being a rooftop garden. A scale factor of 2/3rd has been applied to the prototype structure resulting in an inter-storey height of 2 m and a building footprint of 4 m by 3 m. In order to evaluate the required amount of mass to be added to the test frame the masses of the prototype building have

been scaled by the factor of 2/3rd observing mass similitude related to the Cauchy-Froude similitude laws. The additional mass required was made up of a combination of concrete blocks and steel hold downs with 12 blocks being spread out across each floor. 50 instruments were placed upon the test structure to evaluate the experimental dynamic behaviour and connection deformations in real time.

The test frame was made from glulam grade GL32h (EN 1995-1-1 2004) and was constructed in the UNIBAS lab in two days by only four workers. The flooring of the building was made from solid glulam panels. All design was performed in accordance with the current version of the Italian design codes (NTC 2008).

2.1 Energy dissipating devices.

During dynamic testing energy dissipation devices were added to the structure in order to add strength and reduce displacements without the increase of accelerations or base shears. The passive hysteretic devices were yielding steel angles and were located at beam-column and column-foundation connections (Fig. 3). The dissipative system was based on the DIS-CAM system (Dolce et al., 2006) developed at the University of Basilicata in Potenza, Italy and consisted of the use of steel angles which are designed to yield in a controlled manner. These angles not only provide dissipative capacity (thus reducing demand) but also significantly contribute to capacity. The angle performance is controlled by milling down of a certain section of an equal steel angle (Fig. 4).



Figure 3. Details of a) beam-column and b) column-foundation connections.

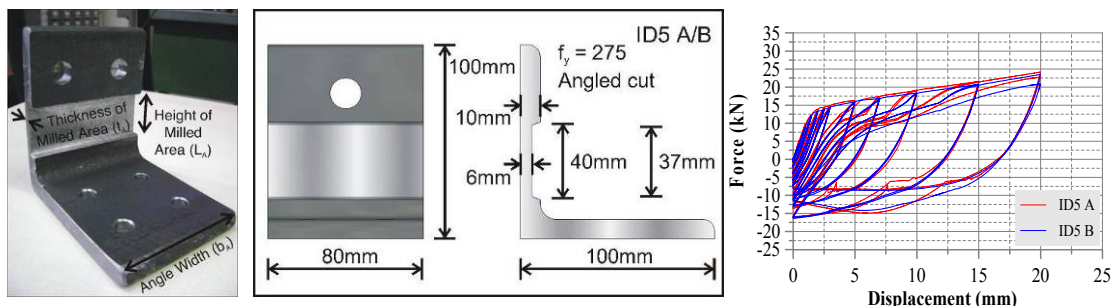


Figure 4. Steel angles device for beam-column joint selected for dynamic testing (with dissipation) and experimental force-displacement device characterisation.

For more information regarding dissipative angle reinforcing refer to Di Cesare et al. (2013).

2.2 The shaking foundation.

The testing apparatus consisted of a shaking foundation (Fig. 5) present in the laboratory of the University of Basilicata. The foundation had a single degree of freedom in the N-S direction and consisted of a steel frame made up of HEM300 structural steel sections.



Figure 5. a) Levelling of foundation sliders and b) Shaking foundation in UNIBAS lab.

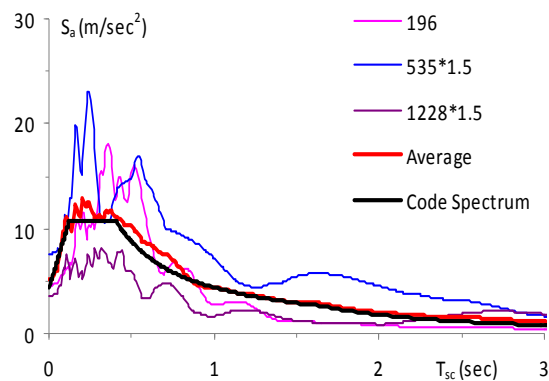
The foundation was situated upon 4 SKF frictionless sliders with one each situated under the four columns. These sliders sat upon a series of levelling plates set upon grout-pads to ensure that a system with a coefficient of friction of less than 1% was obtained. The table was displacement controlled and therefore seismic input was supplied as table displacement over a specific time interval.

2.3 Seismic inputs

The testing input was a set of 7 spectra compatible earthquakes selected from the European strong-motion database. The code spectrum used to select the set of seven earthquakes was defined in accordance with the current Eurocode for seismic design (EN 1998-1:2003 2003) giving a PGA for the design spectrum of $a_g = 0.44g$ (Soil class B – medium soil). In order to match the real acceleration inputs to the code spectrum it was necessary to scale four of the earthquake ground motions (001228x, 000535y, 000291y and 004673y). A smaller set of three ground motions was selected which provided the best representation of the design spectra as shown on the left of Table 1. This smaller set of seismic intensities was progressively increased until the design performance criterion was achieved.

Table 1. Characteristics of selected earthquakes and comparison with the code spectrum.

ID Code	Location	PGA g
001228x	Izmit, Turkey	0.357
000196x	Montenegro, Serbia	0.454
000535y	Erzican, Turkey	0.769
000187x	Tabas, Iran	0.926
000291y	Camp. Lucano, Italy	0.264
004673y	South Iceland	0.716
004677y	South Iceland	0.227



3 NUMERICAL MODELLING

From the conception of the post-tensioned jointed ductile connection it has been clear that the nature of the controlled rocking mechanism lent itself well to the use of a lumped plasticity approach in modelling. This approach combines the use of elastic elements with springs which represent plastic rotations in the system. Recent studies have also recognized the importance of modelling and accounting for the elastic joint rotation in the calculation of rocking connection rotation. Therefore a rotational spring was added in the joint panel region.

This method of modelling was used for the prediction of the structural behaviour of the test frame. The specimen (Fig. 6) was modelled considering rotational springs to predict the moment rotation response

and the effects of dissipation of the post-tensioned beam-column joints and a multi-spring column-foundation interface to match the structural rocking at the base of the frame. Rotational springs were calibrated against the design procedure for the moment calculation of hybrid post-tensioned timber frames presented in the post-tensioned frame design guidelines (Structural Timber Innovation Company Inc. 2013). Modelling used two non-linear finite element codes, SAP2000 and RUAUMOKO. Post-tensioning was represented using tri-linear elastic elements for both models with bounded Ramberg-Osgood and Buoc-Wen rotational spring models used to represent the steel dissipative angles in the RUAUMOKO and SAP2000 model respectively (Di Cesare et al., 2013).

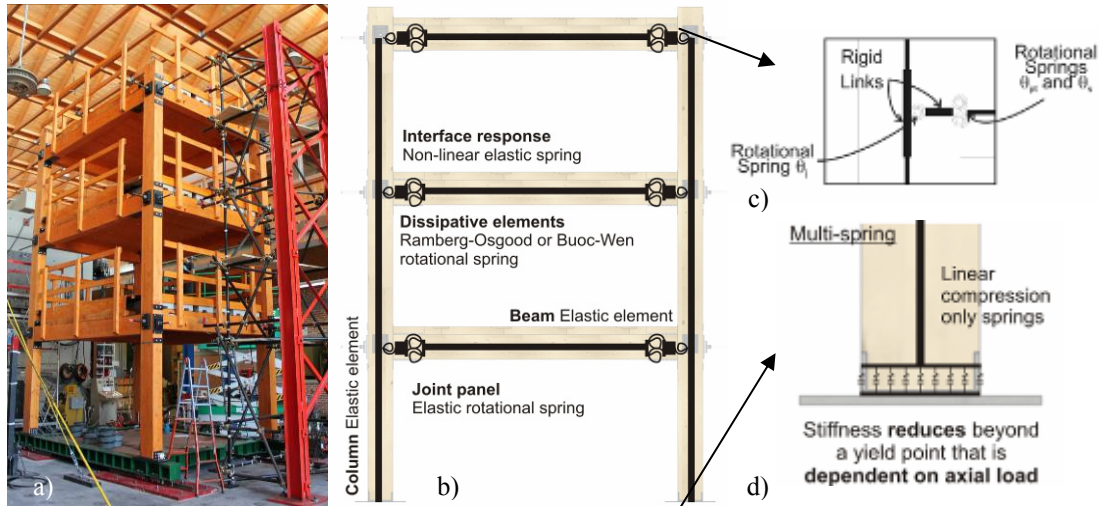


Figure 6. a) Experimental frame constructed in UNIBAS laboratory, numerical model of b) test frame, c) beam-column joint and d) column/foundation interface.

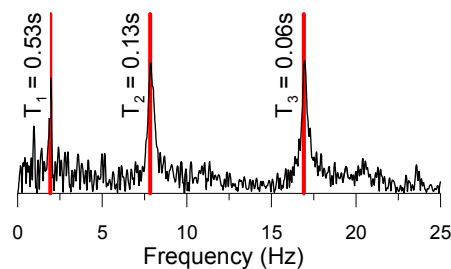
4 EXPERIMENTAL RESULTS

4.1 Dynamic model identification

Dynamic model identification testing was carried out in order to find the natural frequencies of vibration considering two different excitation sources: hammer impact excitations and sine-sweep ground motion. In Table 2 the first three periods of the frame, considering the hammer impact test response on the third floor of the structure with dissipative devices, are compared with the 2-D SAP2000 numerical predictions. The numerical blind predictions ($T_{i,num}$) for SAP2000 model matched well with experimental results $T_{i,exp}$

Table 2. Comparison between dynamic experimental behaviour and SAP2000 blind predictions.

Direction	Mode	$T_{i,exp}$ (sec)	$T_{i,num}$ (sec)
Transl. X	1	0.53	0.53
Transl. X	2	0.13	0.12
Transl. X	3	0.06	0.05



Knowing the fundamental frequency of the structure the spectral acceleration at the fundamental frequency of the three principal ground motions were identified and compared against the design value of $S_A(T_1) = 0.84g$. These were 0.49g (58%), 1.58g (188%) and 1.69g (2.01%) for ground motions 001228x, 000196x and 000535y respectively. This showed that at the fundamental period of the structure the earthquake demand was on average higher than design.

4.2 Shaking table tests.

A series of shaking table tests with increasing PGA levels were performed both with and without

dissipative devices. A summary of the base shear versus first floor drift response is shown in Figure 7. Figure 8 shows photos of the maximum response of the structure during testing which occurred during testing with the dissipative reinforcing subjected to ground motion 000535y at 100%.

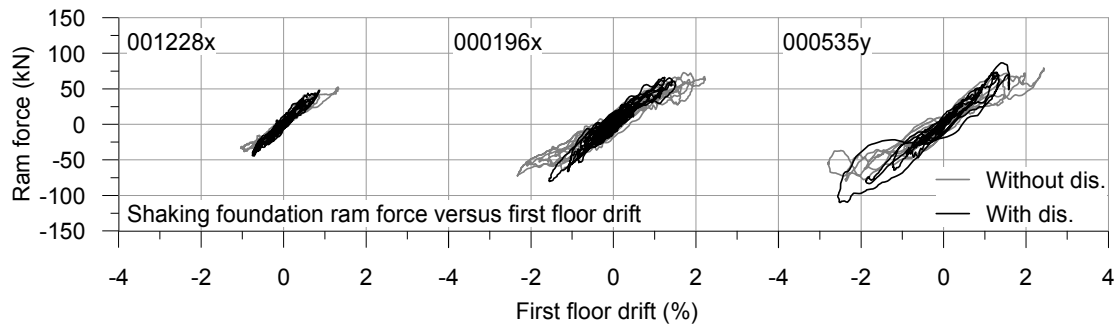


Figure 7. Shaking table force vs first floor drift for the model with and without dissipation (PGA75%).

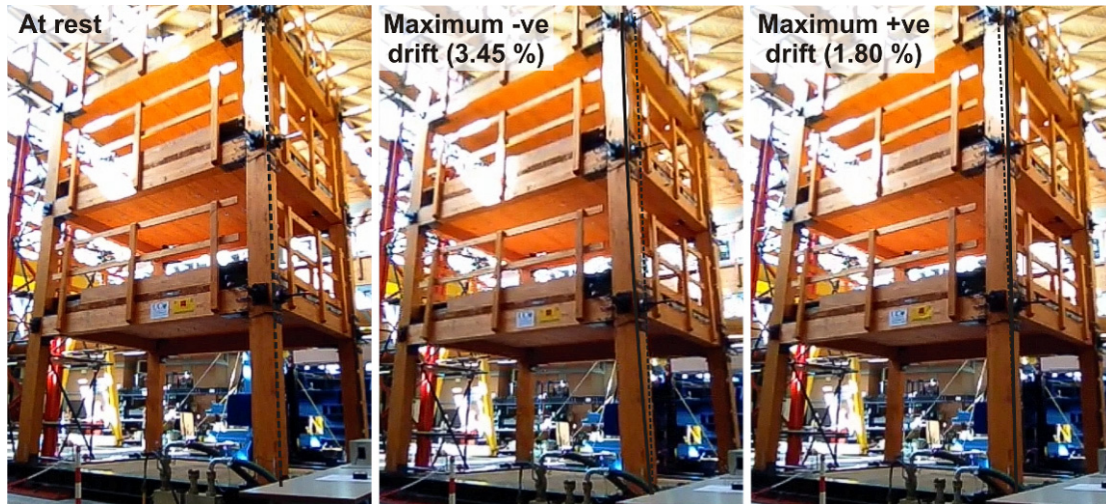


Figure 8. Photos showing maximum positive and negative drift response of the structure, during testing with dissipation, 000535y at 100%.

Figure 9 shows the maximum average (across input 001228x, 000196x and 000535y) drift of the three levels of the test structure for the configurations with and without dissipative reinforcing. The figure clearly shows that under dynamic loading the addition of the dissipative angle reinforcing reduced maximum drifts under the same input acceleration.

The figure also shows that the two systems responded very similarly in terms of drift for low levels of the seismic action. This indicated that the presence of dissipative reinforcing will not impact on serviceability level response. This is due to the fact that before this point gap opening has not occurred and the dissipative reinforcing remains nominally loaded. Following the PGA50% intensity level the response of the frame differed with a rapid increase of drift levels in the case without dissipation while for the dissipative case this rapid increase occurred following PGA75% testing. The presence of the steel dissipative angles led to a 32% decrease in average first floor drift between testing with and without the dissipative angles at PGA75%.

Figure 9 also shows the average maximum 3rd floor accelerations for the two test configurations with increasing percentages of PGA. At higher levels of PGA no significant increase in maximum average floor acceleration due to the addition of the dissipative reinforcing existed despite the reduction in maximum average drift.

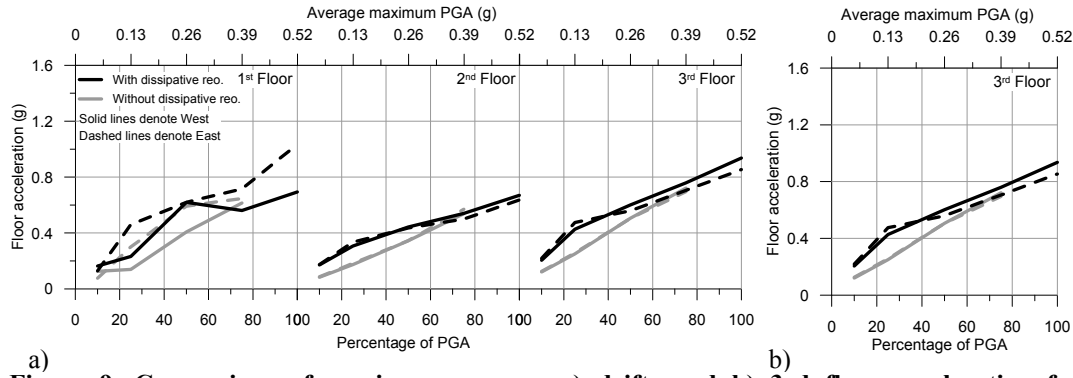


Figure 9. Comparison of maximum average a) drifts and b) 3rd floor acceleration for test frame increasing PGA levels.

4.3 Numerical comparisons.

The experimental results of the dynamic testing are compared in Figure 10 with the SAP2000 and RUAUMOKO multi-spring base model considering the 3rd floor displacements for the 000196x ground motion case with an intensity level of 75% of the design PGA.

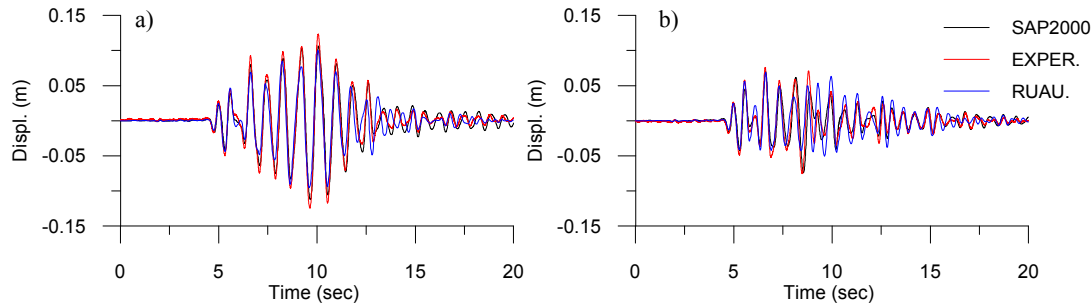


Figure 10. Comparison between SAP2000, RUAUMOKO and testing results a) without and b) with dissipative reinforcement.

As shown in Figure 10 the numerical predictions provided an accurate representation of the experimental performance for both models. Comparisons between the RUAUMOKO and SAP2000 non-linear time history analysis showed that the numerical results had the same trend in time with a small difference in maximum values (less than 10%) for both the configurations with and without dissipation.

Figure 11 shows the SAP2000 and RUAUMOKO results in terms of a) maximum inter-storey drift (MID) and b) maximum base shear (MBS) averaged across the 3 principal input accelerograms (000196x, 000535y and 001228y) compared with the experimental tests with and without dissipative steel angles corresponding to an intensity level of 75% of the design PGA.

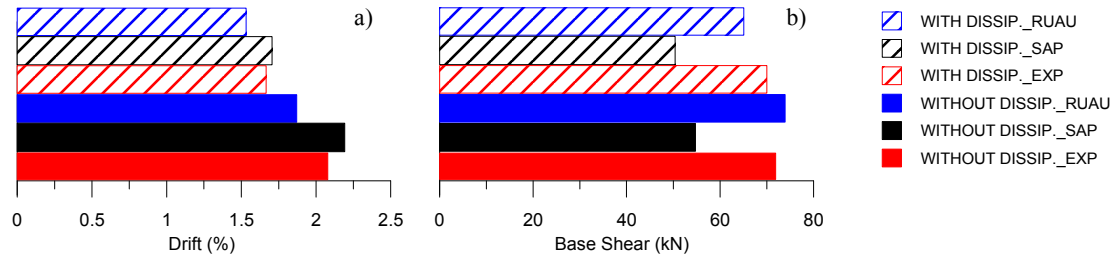


Figure 11. Average of a) MID and b) MBS provided by SAP2000 and RUAUMOKO modelling compared with experimental outcomes with and without dissipation.

Comparisons between SAP2000 and RUAUMOKO analyses show that both programs provided adequate prediction of first floor drift with the SAP2000 programme providing more accurate results. Comparison of the maximum base shear shows that SAP2000 did not accurately predict values (errors in the range of 30%) however RUAUMOKO did capture values well.

5 CONCLUSIONS

An extensive dynamic testing campaign has been performed on a multi-storey post-tensioned timber building in the laboratory of the University of Basilicata in Potenza, Italy in collaboration with the University of Canterbury in Christchurch, New Zealand. The project aims to develop the innovative post-tensioned timber (PRES-LAM) concept by extending its application to glulam timber with innovative methods of dissipative reinforcing. The experimental model has been tested both with and without the addition of steel angles which are designed to yield at a certain level of drift. Testing results have shown the effectiveness of providing capacity with dissipative reinforcing in reducing displacement without increasing acceleration (and therefore base shear). At 75% of maximum PGA, the presence of the steel dissipative angles reduced the maximum average first floor drift by 1.5 times without creating increased third floor peak accelerations. This paper has also presented comparisons between experimental and numerical results using two non-linear finite element programmes: SAP2000 and RUAUMOKO. NTHA numerical results were all within 30% of the experimental results.

6 ACKNOWLEDGEMENTS

The authors would like to acknowledge funding from the Structural Timber Innovation Company (STIC) and FederlegnoArredo (FLA).

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