Abstract: - This paper describes the main results of shaking table testing performed on a post-tensioned timber frame building equipped with an advanced damping system. This experimental campaign, carried out in the structural laboratory of the University of Basilicata in Potenza, Italy, is part of a series of experimental tests in collaboration with the University of Canterbury in Christchurch, New Zealand. The specimen is 3-dimensional, 3-storey, 2/3\textsuperscript{rd} scale and is made by using post-tensioned timber frames in both directions. During the testing programme, the specimen was tested both with and without the addition of 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. This paper discusses the main results of the experimental testing and presents comparisons with numerical outcomes using two non-linear finite element codes: SAP2000 and RUAUMOKO.

Key-Words: - Shaking table test, Timber Buildings, Damping system, Post-Tensioning, Passive control, Steel dissipating devices, Pres-Lam technology.

1 Introduction
This paper describes the main results of shaking table testing performed on a multi-storey post-tensioned timber frame building equipped with an advanced damping system. The study evaluates the feasibility of applying jointed ductile post-tensioning technology, originally conceived for use in concrete structure [11], to glue laminated timber (glulam). The aim of the project is to evaluate the seismic performance of the system and further develop the system for use in multi-storey timber frame 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 [6, 12]. 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. The structural response of a post-tensioned timber frame centres on the moment-rotation response of its connections. In order to design all structural characteristics a certain starting point must be selected as the design drift ($\theta_d$). From this point all of the elastic contributions of the frame must be subtracted (beam rotation, column rotation, joint panel rotation, interface rotation). Having obtained the rotation to be imposed at the beam-column joint the Modified Monolithic Beam Analysis (MMBA) method is used in order to calculate the moment capacity of the
rocking joint. From this, the required value of initial post-tensioning and steel reinforcing can be defined and the key to this type of system is made up of the ratio $\beta$ between the moment resistance provided by the post-tensioning and the moment resistance provided by the dissipation (Fig 1).

![Moment response with varying levels of the parameter $\beta$](image)

**Fig. 1.** Moment response with varying levels of the parameter $\beta$

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 Stage One of PRES-LAM project a full-scale beam-column joint was designed, fabricated, constructed and tested at the Structural Laboratory of UNIBAS. This experimental programme was completed midway through 2011 providing excellent results [13] and began to answer key questions regarding system performance. During testing the application of the post-tensioned timber concept to glulam timber was confirmed and the system displayed the same excellent performance under static loading as when the system was employed with laminated veneer lumber (LVL). In the current stage of the project a 3-dimensional, 3-storey timber structure (Figg. 2 and 3) 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 different values of post-tensioning and steel reinforcing moment capacity contributions were investigated. 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, the testing set-up of the experimental model and the testing results with and without dissipation. Finally, comparisons between experimental outcomes and SAP2000 and RUAMUKKO numerical predictions will be also shown.

### 2 Testing Structure

The prototype structure is three stories in height and has a single bay in both directions. The interstorey height of the prototype building is 3 m and the frame footprint is 6 m by 4.5 m. The building has been designed to represent an office structure (live loading $Q = 3$ kPa) with the final floor being a rooftop garden. The flooring of the building is made by solid glulam panels. The lateral resistance of the building is governed by seismic loading. The test frame is made from glulam grade GL32h (EN 1995-1-1 2004) and has been constructed in the UNIBAS lab in the brief time of two days by only four workers. All design has been performed in accordance with the current version of the Italian design code [7]. A scale factor of $2/3$ has been applied to the prototype structure resulting in an interstorey height of 2m and a building footprint of 4m by 3m. 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 $2/3$ observing mass similitude related to the Cauchy-Froude similitude laws. The additional mass required is made up of a combination of concrete blocks and steel hold downs with 12 blocks being spread out across each floor. 50 sensors have been located in the test structure to evaluate the experimental dynamic behaviour and connection deformations in real time. For more information regarding the specimen design, connection detailing and testing set-up please refer to [10].

The shaking table tests were carried out using the testing apparatus (Fig4) of the seismic 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.

![Frame assembling in UNIBAS lab](image)
The foundation was driven by an MTS 244.41 dynamic actuator characterized by a capacity of ± 500 kN and ± 250 mm stroke. The actuator was fixed to a hinge at the base of the foundation and pushed against the 6 m thick strong wall used during the beam-column test programme. Pressure for the actuator was provided by 3 MTS SilentfloTM 505-180 hydraulic pumps. The foundation was situated upon 4 SKF frictionless sliders (model LLR HC 65 LA T1) 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.

2.1 Energy dissipating devices
During dynamic testing energy dissipation devices have been added to the structure in order to add strength and damping and reduce displacements without increasing of accelerations or base shears. This passive hysteretic devices are made by yielding steel angles and have been located in beam-column and column-foundation connections (Fig. 5).

The dissipative system is based on the DIS-CAM system [3] developed at the University of Basilicata in Potenza, Italy and consists of the use of low carbon steel angles designed to yield in a controlled manner. These angles not only provide dissipative capacity (thus reducing demand) but also significantly contribute to capacity. An extended experimental campaign has been performed at UNIBAS in order to define the angle forms to be used in the dynamic experimental model. Following this two different methods of creating a concentrated yield area were selected both based on the concept of creating a controlled zone of concentrated yielding. The first option (taken from a section of square hollow section tube) involves the removal of two holes and the second of these involves the milling down of a certain section of an equal steel and. Milled steel angle ID5 configuration
has been chosen for dynamic testing with dissipation adopting the angled cut as particular form of transition. This type of transition proved to be successful and simple and is now recommended for all milled angle manufacture.

For more information regarding the experimental campaign, please refer to [1].

Seismic loading during testing has been monodirectional applied along the north-south axis of the building. The seismic intensities were progressively increased until the design performance criterion was achieved. The code spectrum was defined in accordance with the current Eurocode for seismic design (EN 1998-1:2003) having a PGA of \( \text{ag}= 0.35 \) and a soil factor of \( S = 1.25 \) (Soil class B – medium soil) giving a PGA for the design spectrum of \( \text{ag}= 0.4375 \). The elastic response spectra of three accelerograms used in blind numerical predictions SAP 2000 and RUAUMOKO are shown below with their Average and Code Spectrum.

### 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 procedure can be applied to the design of a timber hybrid connection provided a few simple considerations are made. This method of modelling has been used in the predictive modelling of the structural behaviour under the planned input loading. The specimen was modelled (Fig. 8) 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 rocking/foundation interface to match the 
structural rocking movement. Rotational springs 
have been calibrated against the design procedure 
for the moment calculation of a hybrid joint 
presented in Appendix B of the New Zealand Code 
for the Design of Concrete Structures [8]. 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 
[2].

3 Shaking Table Tests
In the first series of tests, carried out on July 2013, 
the structure was first tested with the addition of the 
dissipative reinforcing angles. Upon completion the 
angles were removed and testing without dissipation 
was performed.

Figure 9 shows the maximum average 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%. The 
point of gap-opening is a function only of the 
amount of initial post-tensioning across the 
interface, however in a dissipative reinforced 
system, following gap-opening the joint also has the 
stiffness and strength provided by the angles in 
order to resist rotation leading to later onset of non-
linear behaviour.

Figure 9 shows also the average maximum floor 
accelerations for the two test configurations with 
increasing percentages of PGA. The figure also 
shows that as levels of PGA% increased the 
differences in floor acceleration between the case 
with and without dissipation decreased.

![Graph showing maximum average drifts and acceleration for test frame increasing PGA levels.](image-url)
For low levels of PGA% a slight increase in floor acceleration is observed. It is likely that this was due to the increased stiffness of the structure before the initial slipping of the floors and the failure of the column base connection. These two factors led to a reduced building stiffness as evidenced by a higher building period. The base shear response of the structure with and without dissipative reinforcing is also shown in Figure 10. This was calculated using the accelerations and the model masses. Although a load cell was placed on the dynamic actuator these readings included the weight of the shaking foundation and provided higher values than what was actually present during testing. As expected the base shear display the same general trend as the accelerations presented above. A maximum average value of 97 kN was recorded for testing case with dissipation corresponding to 100% of PGA intensity which was similar to the design level base shear.

During the first series of testing, carried out on 2013, some slipping of the dissipative connection was observed. This reduced the effectiveness of the reinforcing by decreasing both stiffness and dissipative capacity. Although the slipping shown is approximately 3 mm this represented almost half of the expected reinforcing displacement and was 6 times the yielding displacement. It is likely that this fact led or at least was a contributing factor to the increased drifts and displacements. Slipping between the base of the dissipative reinforcing and the connection plate also explains why the dissipative loops were low in spite of the significant levels of gap opening. It is unclear the degree to which this slipping occurred during testing with only one of the connections being recorded. In any case, despite this anomaly, the system demonstrated its overall effectiveness and thus its robustness.

The final study of the dynamic test results involves the evaluation of the elastic and inelastic damping of the frame. During a seismic event the presence of damping reduces demand on a structure. The total equivalent viscous damping of a structure \((\xi)\) is made up of two sources: the elastic \((\xi_{el})\) and the hysteretic damping \((\xi_{hyst})\). Damping values are evaluated and compared in this section.

Elastic damping is used to introduce damping not captured by the hysteretic model represented by the codified reduction methods. Many methods can be used in order to evaluate the elastic damping of a building. During the analysis of the test structure, the half-power bandwidth (HPB) method [4] was used. This method estimates the damping using the frequency range, in combination with a Welch Fourier analysis [14]. The HPB method returns significant results in the analysis of a stationary system (i.e. no significant non-linear response) therefore it has been applied using the forced vibration hammer identification testing. In the application of the HPB method, firstly, the amplitude of each (in this case the first) natural frequency is obtained. Values of \(\xi_{el} = 1.49\%\) and \(1.84\%\) were calculated for tests with and without dissipation respectively.

Test configuration without reinforcing showed slightly elevated elastic damping likely due to the reduction in stiffness created by the series testing with dissipation. It is important to note however that an experimental model is without many of the sources of elastic damping present in a normal structure (partitions, cladding etc.).Values were therefore expected to be lower than in a real post-tensioned timber structure which contains these elements.

Although it was not possible to directly compare this value to the design values and assuming a linear trend to continue, a value of approximately 91 kN (similar to the performance point) could be obtained also for testing case without dissipation.
As with elastic damping, several methods are available to evaluate inelastic damping [5]. The evaluation of hysteretic damping ($\xi_{hyst}$) during dynamic testing is complicated by the forced nature of the system response. In order to have hysteretic damping gap opening and dissipative reinforcing, yield must occur. With increased displacement beyond yield (i.e. increased ductility) the equivalent viscous damping of a post-tensioned timber structure also increases. During seismic response therefore, damping increases with stronger ground motion. Test without dissipation did not contain the use of dissipative reinforcing at the beam-column joint indicating that the timber system itself is capable of providing nominal amounts of hysteretic dissipation during strong motions [9]. The total response of the system without dissipation was very similar to the response of model with dissipation ($\xi_{hyst}=2.8\%$ for PGA75%) tending to indicate that the damping characteristics of the two configurations are very similar. This is not unexpected as the dissipative reinforcing will only provide damping during the maximum frame response cycle of which few occur during the total frames responses. It can be seen that only one clear flag shaped hysteretic loop occurred during all tests. Performing the area EQV analysis method suggested by [5], the equivalent viscous damping value of this loop was $\xi_{hyst} = 7.8\%$.

The experimental outcomes of the dynamic testing are compared in Figure 11 with the SAP2000 and RUAUMOKO multi-spring base model considering the 3rd floor displacements for 00196x test case with an intensity level of 75% of the design PGA. As shown in the figure, numerical predictions provided an accurate representation of the experimental performance for both models. Comparisons between RUAUMOKO and SAP2000 non-linear time history analysis have shown that both codes provide adequate prediction of first floor drift with the SAP2000 programme providing more accurate results. Comparison of the maximum base shear shows that SAP2000 does not accurately predict values either with or without reinforcing however RUAUMOKO does capture values well in both cases. As can be seen NTHA confirmed the interstorey drift and base shear in the structure reducing in amplitude by 1.5 and 1.2 factor respectively when additional steel angle devices are introduced.

Figure 12 shows SAP2000 and RUAUMOKO multi-spring base model results in terms of a) maximum interstorey drift (MID) and b) maximum base shear (MBS) averaged across 3 input accelerograms compared with the experimental tests with and without dissipative steel angles corresponding to an intensity level of 75% of the design PGA. Comparisons between SAP2000 and RUAUMOKO NTHA analysis show that both codes provide adequate prediction of first floor drift with the SAP2000 programme providing more accurate results. Comparison of the maximum base shear shows that SAP2000 does not accurately predict values either with or without reinforcing however RUAUMOKO does capture values well in both cases. As can be seen NTHA confirmed the interstorey drift and base shear in the structure reducing in amplitude by 1.5 and 1.2 factor respectively when additional steel angle devices are introduced.
4 Conclusion
An extensive dynamic testing campaign has been performed on a multistorey post-tensioned timber building in the laboratory of the University of Basilicata in Potenza, Italy. The aim of the project, in collaboration with the University of Canterbury in Christchurch, New Zealand is to develop the innovative post-tensioned timber concept (under the name PRES-LAM) by extending its application to glulam timber with dissipative steel angle devices. The experimental model has been tested both with and without the addition of steel angles which are designed to yield at a certain level drift. This paper confirmed the validity of MMBA procedure used in order to calculate the moment capacity of the rocking joint selecting a certain starting point which is taken as the design drift (θd). Testing results have been shown in this paper proving the effectiveness of post-tensioned system adding dissipative steel devices in reducing seismic effects if compared to that of the system without ones. The presence of the steel dissipative angles allow a maximum reduction of the seismic behaviour in the range values of 1.2-2 times in comparison with PT only configuration. By varying post-tensioning force and the contribution of steel dissipation devices in beam-column joints and column/foundation interface a designer can control the global seismic response of the system. NTHA numerical outcomes using SAP2000 and RUAUMOKO matched experimental results confirming the effectiveness of dissipation with hysteretic steel angles.

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References: