SHAKING TABLE TESTING OF A MULTI-STOREY POST-TENSIONED GLULAM BUILDING: PRELIMINARY EXPERIMENTAL RESULTS

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ABSTRACT

This paper describes the results of preliminary shaking table testing performed on a post-tensioned glulam framed 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 is 3-dimensional, 3-storey, 2/3rd scale and is made up of post-tensioned timber frames in both directions. During the testing programme the specimen was tested with and without the addition of dissipative steel angle reinforcing which was designed to yield at a certain level of frame drift. These steel angles release energy through hysteresis during movement thus increasing damping. The specimen was subjected to a selection of natural earthquake records with increasing (as % of PGA) levels of seismic loading. This paper briefly discusses the testing set-up and then presents the result of the first phase of experimental testing with and without additional reinforcing.

1 INTRODUCTION

The use of post-tensioning technology [1] to connect structural timber elements enables the design of multi-storey buildings having large bay lengths (8-12m) and reduced structural sections. Developed at the University of Canterbury (UoC) under the name Pres-Lam, Post-tensioned timber uses post-tensioning technology (frequently applied to concrete structures [2]) in order to connect structural timber elements. While the post-tensioning provides desirable recentering properties, the dissipative reinforcing devices allow energy dissipation from the system as well as increasing moment resistance. During lateral movement, controlled rocking occurs at the beam-column or column-foundation interface which provides flag-shaped hysteretic behaviour.

An extensive dynamic experimental testing programme is being performed in the structural laboratory of the University of Basilicata (UNIBAS) in Potenza, Italy. This work is part of a collaborative experimental campaign between UNIBAS and UoC. The aim of the project is to further study the performance of post-tensioned frame systems and improve understanding of their dynamic performance.

In the current stage of the project a 3-dimensional, 3-storey timber structure has been dynamically tested in the UNIBAS laboratory. During the experimental campaign the size of the structural members, the building layout and the mass were not altered, however different configurations of the testing structure with and without additional energy dissipation devices have been considered. This paper will first briefly describe the detailing and testing set-up of the experimental model, following which, preliminary testing results will be presented and discussed in order to evaluate the impact of the design choices have on the structural dynamic response. Finally focus is placed on the elastic and inelastic damping of the frame. The non-linear numerical modelling of the frame is presented in a companion paper in the conference proceedings [3].

2 DESIGN CONCEPT

The moment capacity of a post-tensioned glulam beam-column joint was experimentally studied during Stage One of this project, as reported in [4]. During design,
the rotation of the joint, of which the moment capacity is a function, must be calculated from the total building drift \( (\theta_t) \). This total drift is made up of five separate rotation contributions \([5]\) as shown in Figure 1. The first two rotation contributions comprise of the elastic rotations of the beam \((\theta_b)\) and column \((\theta_c)\). Due to the low shear modulus of timber, the large axial forces in the beams induced by the post-tensioning result in large elastic shear deformations in the joint panel which must also be accounted for (joint panel, \(\theta_j\)). The final two rotation contributions make up the total rotation of the connection \((\theta_{con})\). These two contributions are defined as the interface rotation \((\theta_{int})\) and the gap rotation \((\theta_{gap})\) and are calculated and act separately. The interface rotation acts before the decompression point of the beam and the gap rotation occurs after decompression.

Following decompression the Modified Monolithic Beam Analogy (MMBA) is used \([6]\). This method, developed for post-tensioned jointed concrete structures, draws an analogy between the deformations and stresses in a hybrid joint and those occurring in a standard concrete connection and involves the imposing of a gap rotation \((\theta_{gap})\) and the initial estimation of a neutral axis value \((c)\). This procedure can be simply applied to the design of a timber hybrid connection provided a few simple considerations are made \([7]\). Using the design procedure, the forces in the post-tensioning tendon, compression in the timber, and the force in any dissipative reinforcing element are calculated. Force equilibrium is then checked and if not satisfied a new value of neutral axes \(c\) is selected. Once force equilibrium is verified the moment contributions are added in series about a common point. Key to connection performance is the ratio \(\beta\) between the moment resistance provided by the post-tensioning \(M_{pt}\) and by the additional dissipation \(M_{dis}\) for a total moment resistance provided by the joint equal to \(M_t = M_{pt} + M_{dis}\) (Figure 2). Clearly, during design, this choice affects both the damping and moment capacity of the system and therefore changing this value will have a direct impact on both capacity and demand.

![Figure 1: Deformation in a post-tensioned timber frame](image1)

![Figure 2: Moment response with varying levels of the parameter \(\beta\)](image2)

### 3 EXPERIMENTAL MODEL

The prototype structure is three stories high and had single bays in both directions. The design has been performed in accordance with the current version of the Italian and European design codes \([8]\). The test frame is made from glulam (grade GL32h) and was designed to represent an office structure (live load of \(Q = 3\) kPa for level I and II) and has a rooftop garden (\(Q = 2\) kPa). A scale factor of 2/3rd has been applied to the prototype structure obtaining an interstorey height of 2 m and a building footprint of 4 m x 3 m. The section sizes used in the frame elements are 320 x 200 mm, 305 x 200 mm and 240 x 200 mm for columns, primary and secondary beams, respectively. The reinforcing was added to the base of the columns and also to the beam-column joints in the frames for the test setup with dissipative steel angles (which are both economical and simple to design and use) in order to increase structural damping under maximum loading. During the experimental campaign the size of the structural members, building layout and mass has not been altered, however two values of \(\beta\) have been used. Testing has been performed under dynamic loading in real time. An overview of the connection details and the experimental model built in the structural laboratory of UNIBAS is shown in Figure 3. More detail of experimental model can be found in \([8]\).

#### 3.1 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 \([8]\). The passive hysteretic devices were located at beam-column and column-foundation connections (Figure 4a and b respectively). The dissipative system was based on the DIS-CAM system \([9]\) 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 dissipative angle performance is controlled by milling down of a certain section of a steel angle. For more information regarding dissipative angle reinforcing please refer to \([10]\).
3.2 Test Setup and Seismic Inputs

The testing apparatus consisted of a shaking foundation present in the laboratory of the University of Basilicata. The foundation has a single degree of freedom in the N-S direction and consists of a steel frame made up of HEM300 sections. The foundation is driven by an MTS 244.41 dynamic actuator which has a capacity of ± 500 kN and a stroke of ± 250 mm. The actuator is fixed to a hinge at the base of the foundation and pushes against a 6 m thick strong wall. Pressure for the actuator is provided by 3 MTS SilentfloTM 505-180 hydraulic pumps. The foundation is situated upon 4 SKF frictionless sliders (Model LLR HC 65 LA T1) with one each situated under the four columns. These sliders sit upon a series of levelling plates which are adjustable to ensure that a system with a coefficient of friction of less than 1% is obtained.

Instrumentation of the structure consisted of a combination of accelerometers, potentiometers and load cells. Fourteen horizontal accelerometers (two at each floor in each direction and one in each direction on the shaking table foundation) were placed on the structure in addition to two vertical accelerometers at the first and third floor. The horizontal accelerometers were placed in pairs on each floor in opposite (South-west and North-east) corners of the structure. The displacements of each floor were measured directly with 2 potentiometers connected to each floor level and an external reference frame. By using a set of 2 potentiometers spaced as far apart from one another as possible any torsional response of the frame was recorded. Tension in the post-tensioning cables was measured directly for 6 of the 12 post-tensioning bars in the structure with three load cells placed in the direction of loading and three in the transverse direction. Local deformations were also recorded across the gap opening. In total 48 channels of data were used to record the real-time performance of the structure. This was assisted by additional video and image equipment.

The testing input consisted of a set of 7 spectra compatible earthquakes selected from the European strong-motion database (Figure 5). The characteristics of these are shown in Table 1. The code spectrum which was used when selecting the records was defined in accordance with the current Italian and Eurocode for seismic design [11-12] having a ground acceleration factor of $a_g = 0.35$ and a soil factor of $S = 1.25$ (Soil class B - medium soil).
giving a PGA for the design spectrum of 0.44. In order to match the real acceleration inputs to the code spectrum it was necessary to scale earthquakes 001228x, 000535y, 000291y and 004673y by 150%. For the sake of brevity this paper discusses in detail the experimental results considering only the inputs ID 000196x and 000535y.

**Figure 5:** Seismic input comparisons with code spectrum

**Table 1:** Characteristics of the selected earthquakes

<table>
<thead>
<tr>
<th>ID Code</th>
<th>Location</th>
<th>Date</th>
<th>MW</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001228x</td>
<td>Izmit, Turkey</td>
<td>17/08/99</td>
<td>7.6</td>
<td>0.357</td>
</tr>
<tr>
<td>000196x</td>
<td>Montenegro, Serbia</td>
<td>15/04/79</td>
<td>6.9</td>
<td>0.454</td>
</tr>
<tr>
<td>000535y</td>
<td>Erzican, Turkey</td>
<td>13/03/92</td>
<td>6.6</td>
<td>0.769</td>
</tr>
<tr>
<td>000187x</td>
<td>Tabas, Iran</td>
<td>16/09/78</td>
<td>7.3</td>
<td>0.926</td>
</tr>
<tr>
<td>000291y</td>
<td>Campano Lucano, Italy</td>
<td>23/11/80</td>
<td>6.9</td>
<td>0.264</td>
</tr>
<tr>
<td>004673y</td>
<td>South Iceland</td>
<td>17/06/00</td>
<td>6.5</td>
<td>0.716</td>
</tr>
<tr>
<td>004677y</td>
<td>South Iceland</td>
<td>17/06/00</td>
<td>6.5</td>
<td>0.227</td>
</tr>
</tbody>
</table>

**4 EXPERIMENTAL RESULTS**

A series of shaking table tests with increasing PGA levels were performed both with and without the dissipative devices. The initial post-tensioning force value $F_{pt,i}$ along the loading direction was set equal to 100kN for both experimental cases. Adding dissipation ($M_{dis}$) without reducing the post-tension moment contribution ($M_{pt}$) had the effect of both increasing capacity and decreasing demand (see Table 2).

**Table 2:** Experimental test model setups

<table>
<thead>
<tr>
<th>Testing setup</th>
<th>$F_{pt,i}$ (kN)</th>
<th>$\beta$</th>
<th>Moment capacity at 2% total rotation</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>$M_{pt}$</td>
</tr>
<tr>
<td>With dissipation</td>
<td>100 kN</td>
<td>0.60</td>
<td>15.4</td>
</tr>
<tr>
<td>Without dissipation</td>
<td>100 kN</td>
<td>1.00</td>
<td>16.7</td>
</tr>
</tbody>
</table>

**4.1 Dynamic Model Identification**

For each experimental model setup, dynamic identification testing was carried out in order to find the first three natural frequencies of vibration by considering hammer impact and sine-sweep ground motions excitation sources. In Figure 6 the Fast Fourier transforms used in order to identify the dynamic characteristics of the frame are shown along with the experimental natural frequencies ($f_i$) and the corresponding periods ($T_i$). These results refer to the hammer impact test response on the third floor of the structure which provided the clearest representation of the structural modes. A slight change in natural frequency between the test series was noted. As the mass remained effectively identical between the two setups, this result implied a reduction in stiffness occurred between the test with and without dissipation.

**Figure 6:** Fast Fourier transforms of dynamic test model for 3rd floor acceleration response under hammer impact

In addition to the analysis performed above, which was carried out before the commencement of the testing series, the fundamental period of the structure was calculated using the third floor accelerations recorded tail from each test. Figure 7 shows a decreasing trend in the dynamic test frames fundamental frequencies during the test sequence. The major changes occurred during larger levels of PGA percentage and specifically during the second (000196x) and third (000535x) tests of each sequence. It is therefore likely that decrease in frequency shown in Figure 6 was not due to a change in the dynamic behaviour of the frame with the removal of the dissipative reinforcing but to the initial slipping and loosening of the flooring panels.

**Figure 7:** Fundamental frequencies of test model with dissipation over testing sequence of earthquake 001228x, 000196x, 000535x

**4.2 Shaking Table Tests**

The structure was first tested (subjected to the full test regime) with the addition of the dissipative reinforcing angles. Upon completion the steel angles were removed and testing without dissipation was performed. During testing cracking noises were heard accompanied by
spikes in both floor acceleration and displacement.

Figure 8 shows pictures of the maximum drift response of the structure with reinforcing. This occurred during 000535y ground motion at 100% of PGA. The maximum interstorey drift of 3.45% was achieved during this test.

Figure 8: Photos showing maximum positive and negative structure deformation during testing with dissipation

4.3 Seismic Results

Figures 9, 10, 11 and 12 compare the response of the experimental model for the configuration with and without the addition of dissipative steel elements when subjected to earthquakes 000196x and 000535y at two different PGA intensities (25% and 75%). These intensities approximately correspond to the damage and ultimate limit states. The figures show the time-history results in terms of relative displacement and acceleration (3rd floor) and interstorey drift (1st floor). The experimental outcomes display clearly the difference in the seismic behaviour of the frame with and without the addition of the steel angles thus proving the effectiveness of dissipative reinforcing. As can be seen, the interstorey drift of the structure reduced in amplitude when additional steel devices were introduced and no significant increase in maximum average floor acceleration was observed. The presence of the steel dissipative angles at high levels of PGA intensity allowed a maximum reduction of the interstorey drift and relative displacements in the order of 1.5 times when compared to the configuration without dissipation. This reduction in drift was accompanied by only a slight increase of acceleration (1.1 times).

Figures 9, 10,11 and 12 also show the shaking foundation ram force versus first floor drift for both test series. These figures clearly confirm that under dynamic loading the addition of the dissipative angle reinforcing reduced maximum drifts under the same input acceleration. The flag shaped hysteretic loop discussed in Section 1 is also evident for testing with the dissipative reinforcing. The time-history outcomes also confirmed the minor alteration of initial frame stiffness at higher levels of PGA intensity. This alteration is noted only between the testing with dissipative reinforcing which was performed first in the sequence. Testing without reinforcing shows no alteration in stiffness. This point further supports the conclusion that this change was related to the onset of slipping between the flooring panels. From these figures the recentering nature of the frame was also seen with little to no residual drift following testing.

Figure 13 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 shows that the two systems responded very similarly in terms of drift for low levels of seismic action. This indicated that the presence of dissipative reinforcing will not impact on serviceability level response. This is due to the fact that under small drifts gap opening will not occur and the dissipative reinforcing remains unloaded. 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.

Following the PGA50% intensity level the response of the frame changed with a rapid increase of drift levels in the setup without dissipation while for the dissipative case this rapid increase occurred following PGA75% testing. The presence of the steel dissipative angles led to an average 32% decrease in first floor drift between testing with and without the dissipative angles at PGA75%. The reduction in drift came from the increased strength of the joint when the dissipative angles are added and the increased system damping which they provided.

Figure 14 shows the average maximum floor accelerations for the two test configurations with increasing percentages of PGA. Figure 14 also shows that as levels of PGA% increased the differences in floor acceleration between the case with and without dissipation decreased. For low levels of PGA a slight increase in floor acceleration is observed: this was due to the increased stiffness of the structure before the initial slipping of the floors as described in Section 4.1.

Finally, Figure 15 displays the base shear response of the structure with and without dissipative reinforcing with increasing PGA intensity. This was calculated using the accelerations recorded and the model masses. As expected the base shear display the same general trend as the accelerations presented above. The evaluation of the maximum average value of the base shear corresponding to 100% of PGA intensity showed that it was similar to the design level for both testing cases.
Figure 9: Experimental outcomes for the model configuration with reinforcing - 000196x seismic input

Figure 10: Experimental outcomes for the model configuration without reinforcing - 000196x seismic input
Figure 11: Experimental outcomes for the model configuration with reinforcing - 000535y seismic input

Figure 12: Experimental outcomes for the model configuration without reinforcing - 000535y seismic input
Figure 13: Comparison of maximum average drifts for test frame increasing PGA levels

Figure 14: Comparisons of maximum average accelerations of test frame at increasing levels of PGA

Figure 15: Maximum base shear of test frame at increasing levels of PGA for with (left) and without (right) dissipation testing
4.4 Visual Inspection of Beam-Column Joint Behaviour

During testing a video camera was placed at one of the beam-column joint interfaces in order to provide a visual record of its performance. Figure 16 shows the beam-column joint, with a focus on the dissipative reinforcement connection during test 000535y at 100% of PGA. This figure shows that slipping of the dissipative connection during testing was occurring during testing. This slipping was related to the necessity to create holes in the dissipative reinforcing angles which were larger than the fixing blots (+2mm on the radius). This was done to enable the angles to be placed without specific sizing of each individual angle. Although the slipping was only approximately 3 mm, this represented almost half of the expected reinforcing displacement and was 6 times the yielding displacement. The effectiveness of the additional damping was reduced (experimental $\beta_{\text{exp}} = 0.7$ - 0.8) with respect to the design values and consequently displacements recorded during testing were larger than expected. Slipping between the base of the dissipative reinforcing and the connection plates also explains why the dissipative loops were lower than expected in spite of the significant levels of gap opening. Despite the slipping phenomena the maximum drift observed during testing was compatible with design drift. These facts displayed the robustness and the redundancy of the post-tensioning timber frame system.

![Image](image1.png)

**Figure 15:** Performance of dissipative angle during test 000535y PGA100% for testing with dissipation at a) maximum negative drift and b) maximum positive drift

4.5 Elastic and Inelastic Damping Evaluation

A further study of the post-tensioned glulam dynamic frame behaviour involved 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 ($\xi_{\text{el}}$) and the hysteretic damping ($\xi_{\text{hyst}}$).

The elastic damping is used to introduce damping not captured by the hysteretic model represented by the codified reduction methods. This damping has a number of sources. It can result from impact damping, foundations and the interaction between structural and non-structural elements. During the analysis of the test structure the half-power bandwidth (HPB) method [12] was used in order to evaluate the elastic damping of a building. This 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. Values of $\xi_{\text{el}} = 1.49\%$ and $1.84\%$ were calculated for tests with and without dissipation respectively. The test configuration without reinforcing showed slightly elevated elastic damping due to the reduction in stiffness created by the testing series with dissipation. It is important to note that an experimental model is lacking 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 glulam structure which contains these elements (normally in the order to 3% [7]).

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. The total average hysteretic damping response of the system without dissipation was very similar to the response of model with dissipation ($\xi_{\text{hyst}} = 2.8\%$ for PGA 75%) indicating that the damping characteristics of the two configurations were very similar. The 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 [13].

The fact that the average hysteretic response of the structure was similar was not unexpected as the dissipative reinforcing will only provide damping during the maximum frame response cycle of which few occurred during the total frame response. It can be seen in Figure 11 that under high 75%PGA loading only one clear hysteretic loop occurred. Performing the area EQV analysis method [14] of this test at 100%PGA provided a damping value of this loop was $\xi_{\text{hyst}} = 7.8\%$. 
5 CONCLUSIONS
An extensive dynamic test campaign has been performed on a 3-dimensional 3-storey, 2/3rd scaled post-tensioned glulam frames model in the laboratory of the University of Basilicata in Potenza, Italy. The model has been tested both with and without the addition of dissipative steel angle reinforcing. Dynamic testing results have shown the effectiveness of providing capacity with dissipative reinforcing in reducing displacement without increasing acceleration (and therefore base shear). The addition of dissipative reinforcing decreased displacements by 32% when the frame was subjected to a series of records at 75% of target PGA. This was accompanied by an 18% increase in base shear and no increase in floor acceleration. The maximum floor acceleration recorded during testing was at the third floor of the structure (0.9 g) and was 1.7 times PGA. Testing has also shown that without additional dissipation accelerations and displacements in this type of structure are elevated leading to the conclusion that even in low seismic areas a minimum amount reinforcing should be applied. The fundamental initial period of the post-tensioned timber frame was independent of the presence of dissipative reinforcing. This was due to the fact that the dissipative reinforcing requires a certain level of displacement to occur to begin working and as such their contribution is not captured by initial stiffness. Alterations in period did occur however due to the onset of slipping in flooring elements. The damping response of the model without dissipation was very similar to the model damping with dissipation. This fact was not unexpected as the dissipative reinforcing only provided damping during the maximum frame response cycle of which few occurred during the total frames response. Slipping between the base of the dissipative reinforcing and the connection plate has been also observed during dynamic testing. This reduced the dissipative capacity of the frame. During testing no damages occurred to the structural elements proving the robustness and redundancy of the post-tensioning timber frame design concept also with the presence of imperfections (slipping of the dissipative connection).

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