Dynamic Testing of Multi-storey Post-tensioned Glulam Building: Planning, Design and Numerical Analysis

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SUMMARY:
The following paper describes the first stage of dynamic testing of a post-tensioned timber building to be performed in the structural laboratory of the University of Basilicata in Potenza, Italy as part of a series of experimental tests in collaboration with the University of Canterbury in Christchurch, New Zealand. During this stage of testing a 3-dimensional, 3-storey post-tensioned timber structure will be tested. The specimen is 2/3rd scale and made up of frames in both directions composed of post-tensioned timber. The specimen will be tested both with and without the addition of dissipative steel angles which are designed to yield at a certain level drift. These steel angles release energy through hysteresis during movement thus increasing damping. The following paper discusses the testing set-up and preliminary numerical predictions of the system performance. Focus will be placed on damping ratios, displacements and accelerations.

Keywords: Post-tensioned Timber, Dynamic Testing, Wooden Frame Building, Dissipation, Numerical Modelling

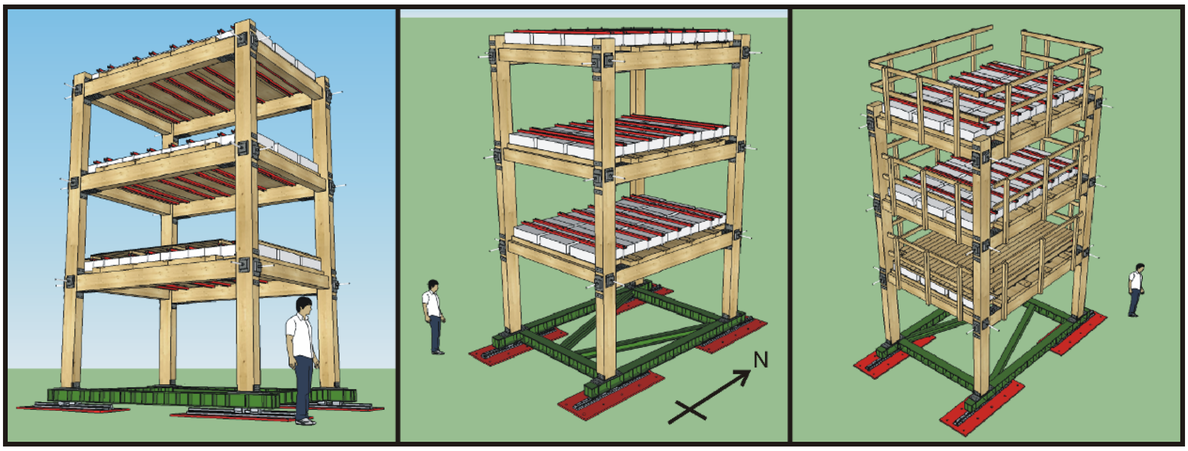
1. INTRODUCTION

This paper presents the design and planned test set-up for the experimental investigation into the seismic performance of a 2/3rd scale three-storey post-tensioned timber frame building in the structural laboratory of the Di.S.G.G in Potenza, Italy. This work is part of a collaborative experimental campaign between the University of Basilicata (UNIBAS), in Potenza, Italy and the University of Canterbury, in Christchurch, New Zealand. The study will evaluate the feasibility of applying jointed ductile post-tensioning technology, originally conceived for use in concrete structures 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 buildings.
The post-tensioned timber concept (under the name PRES LAM) has been developed at the University of Canterbury and extensively tested (beam-column, wall/column-foundation, 3d frame and wall structure) in the structural laboratories of the university (Newcombe et al. 2010b; Palermo et al. 2005; Smith et al. 2007). In Stage One of this 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 and began to answer key questions regarding system performance. During testing the application of the post-tensioned timber concept to glulam timber was confirmed. Testing was performed both with and without dissipative elements and the system displayed the same excellent performance under static and seismic loading as when the system was employed with laminated veneer lumber (LVL) (Smith et al. 2011). 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. A new method of vertical load transfer was also designed and tested, using a hidden steel pipe to resist shear loading without hindering the rocking movement of the beam-column joint. Modelling and design procedures were also implemented and verified (Smith et al. 2012).

An extensive dynamic experimental testing programme, called PRES-GLULAM (PRE-Stressed GLUe LAMinated structural solution), is scheduled to be performed in the Structural Laboratory of the University of Basilicata. This paper will describe the design and proposed construction method of the prototype building. The testing method to be used will then be described followed by a brief description of test set-up and numerical predictions which have been performed.

2. TESTING STRUCTURE

As mentioned the project is based on the PRES LAM concept which uses post-tensioning technology (frequently applied to concrete structures) in order to connect structural timber elements. 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 prototype structure is three stories in height and has single bays in both directions. All design has been performed in accordance with the current version of the Italian design codes (NTC08). The interstorey height of the 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. A summary of vertical loads is described in a following section. The flooring of the building is made from solid glulam panels. Seismic loading governs the building's lateral resistance design.
The test frame (a) is made from glulam grade GL32h. A scale factor of 2/3 has been applied to the prototype structure resulting in an interstorey height of 2 m and a building footprint of 4 m x 3 m. Seismic loading during testing will be mono-directional applied along the north-south axis of the building. Many of the design details developed for Stage One of testing have been used in the design of the frame. The section sizes to be used in the frame are shown in Figure 2.1b with post-tensioning passing through the beams in both directions. The flooring spans in the east-west direction, therefore secondary beams are only required to provide torsional stability.

a) 

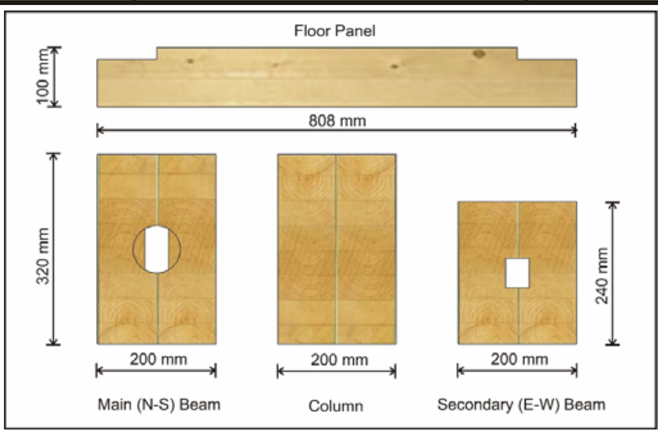
b) 

Figure 2.1. a) 2/3 Scale Post-Tensioned Glulam Test Frame b) Section Sizes Used in Glulam Test Frame

2.1. Test Frame Connection Detailing

This section describes the various connection details that will make up the full test frame. The design of this type of structure can be complicated due to the rocking motion which occurs at the interface of the beam and column members during seismic loading. Rocking creates a concentrated displacement which must be accounted for in design and should not create damage to secondary members, flooring or non-structural elements.

The base of the column (2a) is fitted with a steel shoe which is epoxied into the base of the column and left free to rock on a base plate (which will be used to represent the building foundations in the case of the test building). Four ϕ 20 mm bars of 300 mm length will be used for this connection.

Shear transfer will be achieved using a ϕ 76.1 mm steel tube which extends 15 mm from the steel shoe and slots into a cavity in the base plate. As shown in 2b holes will be drilled and tapped in the steel shoe in order to facilitate the attachment of energy dissipation devices. In order to keep the base of the column as aesthetically pleasing as possible the steel shoe will be recessed into the column.

The beam-column connection in the principle direction will be based on the connection type which was tested during Stage One. Passing through the centre of the beam is a single 26.5 mm diameter bar which will be tensioned to varying values of initial loading throughout the test programme. This is a high strength steel bar with a yield strength f_y = 1050 N/mm^2 and a Young's modulus of 170 kN/mm^2.

As in Stage One, screws will be used to protect the column face with 36 ϕ 8 mm, 80 mm long screws being installed in the column face adjacent to the beam and 30 screws being installed in contact with the post-tensioning back plate (2b). 28 ϕ 8 mm 120 mm long screws will be used to attached each dissipater attachment plate. The various dissipater types are attached to the column though the use of M16 bolts which pass through the width of the column and attach to a backing plate. Where this plate is in contact with the column is also reinforced with 10 ϕ 80 mm long screws. Vertical loading will be transferred through a ϕ 76.1 mm steel tube with extends 66 mm from the beam and sits inside the column.

The beam-column connection in the secondary direction is similar to that in the principle direction. A single ϕ 26.5 mm bar passes through the centre of the beam section. 22 ϕ 8 mm 80 mm long screws are used to reinforce both where the beam and the post-tensioning backing plate meet the column (Figure 2.2b).

As shown above the floor will be made from a series of deep glulam beams oriented flat-wise in order to make flooring panels (Figure 2.2c) running between the principal (N-S) beams. Each 808 mm wide panel is connected to the next using a 20 mm thick strip of plywood which is screwed (ϕ 6 mm 80 mm length) at 150 mm centres. Attachment to the parameter beams will be through the use of 16 pairs of skewed fully threaded screws ϕ 7 mm, 220 mm in length the N-S direction and 15 pairs of skewed partially threaded screws ϕ 6 mm, 240 mm in length in the E-W direction. During design the decision was made to skew these screws due to the significant increase in both stiffness and strength when a screw is placed on an angle.

Skewing of the screws in the direction of loading was also performed in order to mitigate the tearing of the flooring panel during loading. As gap opening occurs the flooring panel near the gap opening is placed in tension as it is attached both to the principle and secondary beams. As seen in Figure 2.2c a section of the floor will be cut out in order to allow the attachment of dissipating devices and it is possible that stress concentrations will occur in the corner of the cut section. By significantly increasing the stiffness of the screw group in the loaded (N-S) direction when compared to those of the secondary (E-W) direction which are skewed in the opposite direction to loading, a fuse is created which will allow the flooring panel to slide over the top of the secondary beam. In order to further reduce stresses in the removed area the screw groups in both directions will begin at least 500 mm from the column.

During testing several passive energy dissipation devices will be added to the structure in order to add strength and reduce displacements without the increase of accelerations of forces. Two methods of passive hysteretic energy dissipation to be added are the (a) *yielding steel angle* device as shown in Figure 2.2d and the (b) *plug and play* axial device shown in Figure 2.2d.

As already mentioned the steel angle devices were applied with great success during Stage One of the project, yielding at low levels of design drift and supplying high levels (up to 15% equivalent viscous damping at the joint, 10% when applied as part of a full frame) of stable hysteretic damping (Smith et al. 2011). The second method of dissipation is an axially yielding bar which is milled down in order to obtain the desired force-displacement performance. This system has also been applied previously to both post-tensioned timber and concrete structures (NCZS 2010). In the design of the additional steel devices the principle parameter used is the re-centring ratio (λ) defined as the moment contribution of the post-tensioning element M_{PT} divided by the contribution of the steel elements M_s. The effect of this parameter will be investigated during the test program.

Additional mass to be added to the frame comes from two sources: the mass due to the scaling of the test frame and the mass due to live loading. As mentioned above the prototype building is an office structure, which also has a rooftop garden. A glass façade (and balustrade in the case of the roof level) in considered to surround the building. The live load values for an office structure are Q = 3 kN/m^2 for the two inhabited levels and Q = 2 kN/m^2 for the open roof.

In order to calculate the required amount of mass to be added to the test frame the masses of the prototype building must be multiplied by the scale factor of $(2/3)^2$. This is related to the use of Cauchy-Froude similitude laws which are to be used in testing. Additional mass required is made up of a combination of concrete blocks and steel hold downs: 12 blocks are spread out across the flooring (Figure 2.2c). For more information regarding building mass please refer to Ponzo et al. (2012)

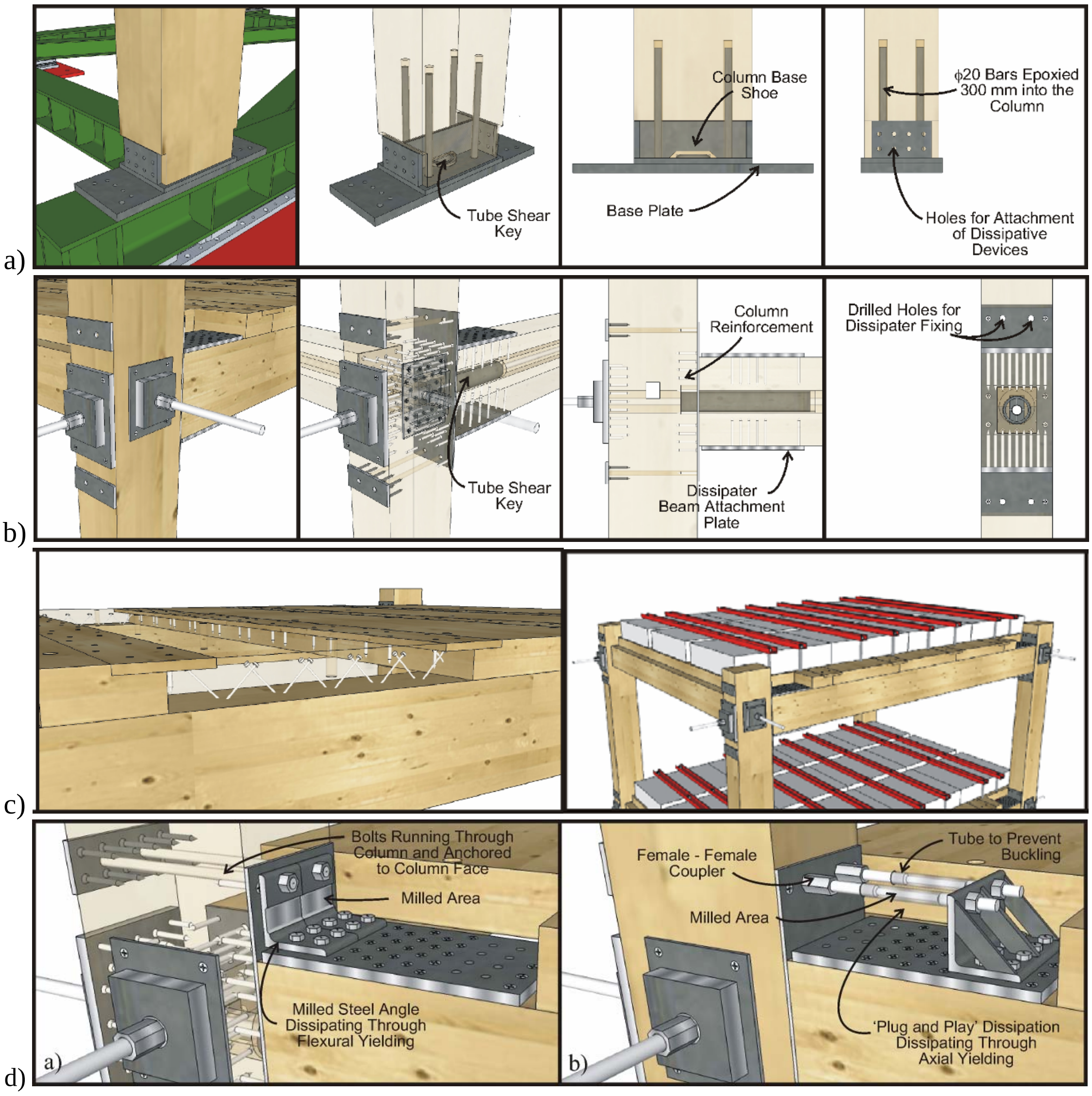


Figure 2.2. a) Post-Tensioned Glulam Frame Column-Foundation Connection b) Post-Tensioned Glulam Frame Beam-Column Connection c) Attachment to Glulam Frame and Timber Flooring with blocks d) (a) Yielding Steel Angle and; (b) 'Plug and Play' Dissipation Devices

2.2.Test Set-up and Seismic Input

Testing will be performed under dynamic loading in real time. This will be done using a shaking foundation testing rig. Due to the fact that the frame is 2/3rd scale, mass similitude must be maintained. This means that additional mass must be added which will also represent the presence of a factored live load. The shaking foundation, added masses and instrumentation will be described in this section.
The testing apparatus consists 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.
The testing input will be a set of 7 spectra compatible earthquakes selected from the European strong-motion database. The characteristics of these spectra are shown in Figure 2.3 along with the code spectrum to which they were compared when considering their suitability. The code spectrum was defined in accordance with the current Eurocode for seismic design having a PGA 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 0.4375.
In order to match the above real acceleration inputs to the code spectrum it was necessary to scale earthquakes 001228x, 000535y, 000291y and 004673y. As the test structure is scaled by 2/3rd the duration of input was divided by the scale factor thus altering input period content $(2/3)^{0.5}$.

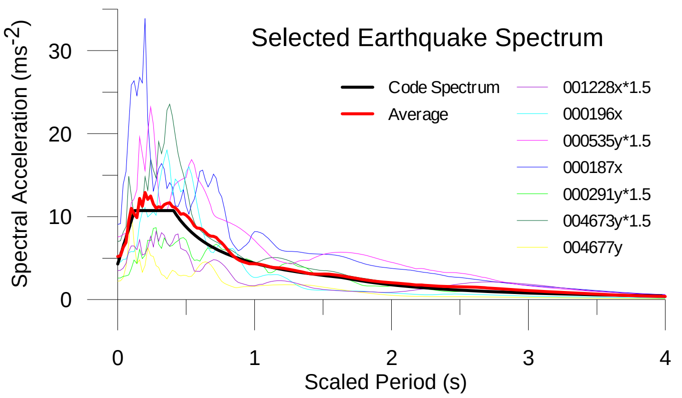


Figure 2.3. Selected Earthquake Input for Shaking Table Testing

3. NUMERICAL MODEL

This section describes the model used during the numerical prediction campaign. The model was made up of a series of rotational springs which represent the moment rotation behaviour of the beam-column joints. These springs were calibrated against the moment rotation design procedure used for post-tensioned timber connections as described in Smith et al. (2012).

3.1. Design Considerations for a Post-tensioned Timber System

During testing several characteristic of the post-tensioned timber concept will be investigated. The key to this type of system is made up of the ratio β , the ratio between the moment resistance provided by the post-tensioning and the moment resistance provided by the dissipation (Figure 3.1). 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. During the experimental campaign the size of the structural members, building layout and mass will not be altered, however different values of post-tensioning and steel moment capacity contributions (thus variations in the value β) will be investigated.

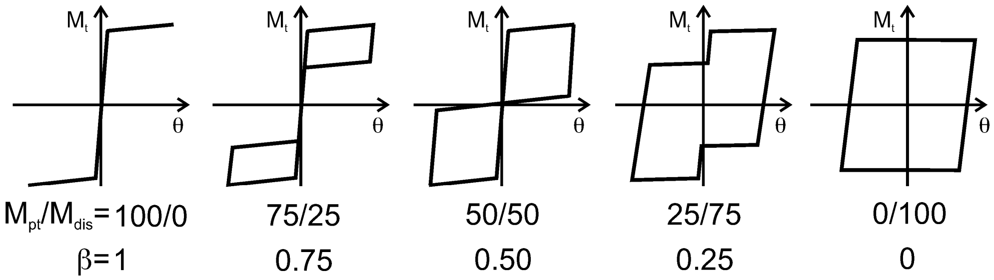


Figure 3.1. Moment Response with Varying Levels of the Parameter β

3.2. The Rotational Spring model

From the conception of the post-tensioned jointed ductile concept 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 (Palermo et al. 2005b). This approach combines the use of elastic elements with springs which represent plastic rotations in the system. This method of modelling has been used in the predictive modelling of the structural behaviour under the planned input loading.
The model was made up of a series of rotational springs used in order to model the moment rotation response of the post-tensioned beam-column joints and the effects of the presence of steel dissipation devices (Figure 3.2a). Recent studies (Cusiel et al. 2010) have also recognised the importance of modelling and accounting for the elastic joint rotation in the calculation of connection rotation. In order to model this a rotational spring was added in the joint panel region (Figure 3.2b).

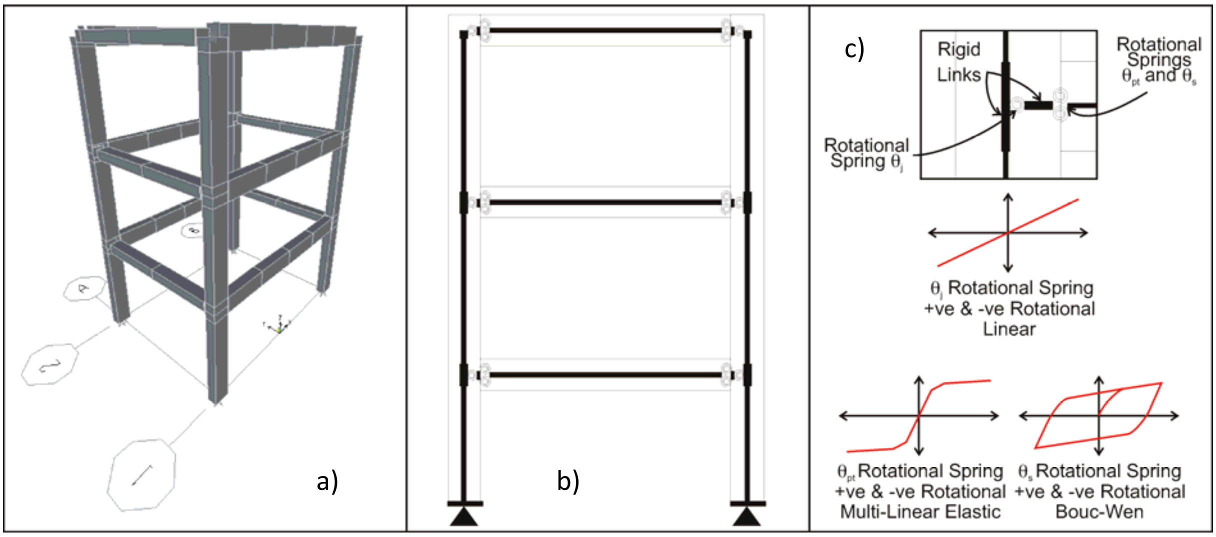


Figure 3.2. a) 3-dimensional numerical model for predictions of b) plan view of Post-Tensioned Timber Frame Model c) Rotational Springs to represent Post-Tensioned beam-column joints, the effects of steel dissipation devices and the joint panel region.

The rotational springs were calibrated against the results obtained for the design procedure used for post-tensioned timber joints. A detailed design procedure for the moment calculation of a hybrid joint has been devised and is presented in Appendix B of the New Zealand Code for the Design of Concrete Structures. This procedure can be simply applied to the design of a timber hybrid connection provided a few simple considerations are made (Newcombe et al. 2008). The model used is shown in Figure 3.2c. Two separate systems were imposed at the joint interface; the first was a multi-linear elastic spring used to model the recentering contribution of the post-tensioning, and this was paired with the bouc-wen hysteretic model used to model the contribution of the steel elements. The linear elastic spring used to replicate joint rotations was given a rotational stiffness based on the moment-rotation relationship of the joint panel using formulas presented in Newcombe et al. (2010a). Rigid links were used in the joint panel region and the beam-column interface and the model was assumed to be pinned at the base of the columns.

4. RESULTS OF NUMERICAL STUDY

As mentioned above the case studies selected were used in order to define the test cases which will be used during the experimental investigations. Cyclic, impulse and time history analyses were performed in order to define total system behaviour and damping characteristics.
During the numerical investigations a series of differing test conditions were considered which are summarised in Table 4.1. These studies were performed in order to define the case studies which will be applied during the testing campaign. Several design hypotheses were made during investigations. These where based upon the selection of: a) Design drift, b) Design moment and c) Recentering ratio β. The seven cases as shown below were studied to varying degrees using non-linear impulse, cyclic and time history analysis techniques.

Table 4.1. Cases Considered During Numerical Analysis

Name	Initial Value of Post-tensioning	β	Moment Capacity at 2% connection rotation		
			M_{pt}	M_s	M_t
Post-tension Only Case					
PT100_1.00	100 kN	1.00	27.21	0.00	27.21
Hybrid Cases (with the addition of steel elements)					
PT100_0.80	100 kN	0.80	27.06	6.35	33.41
PT100_0.66	100 kN	0.66	27.06	13.07	40.13
PT100_0.50	100 kN	0.50	27.06	26.51	53.57
PT 50_0.66	50 kN	0.66	22.35	10.62	32.97
PT 20_0.66	20 kN	0.66	19.51	8.98	28.49
PT 50_0.80	50 kN	0.80	22.35	5.58	27.93

4.1. Results of Cyclic Testing

Cyclic testing was performed by applying a positive and negative triangular force based ramp function to the structure. The results of the analyses are shown in Figure 4.1 in terms of global behaviour of the structure, total base shear and drift evaluated as top displacement divided by the height of the building. In Figure 4.1 the difference in the design selections can be seen. In Figure 4.1 (left) the cases where the initial post-tensioning values were held at 100 kN are shown. Several values of β were then used, meaning that the overall resistance of the system increased so that for the same value of total base shear the drift reached during cyclic loading decreased. As the dissipative devices applied to the structure are displacement dependant all cases displayed a certain amount of hysteretic dissipation.

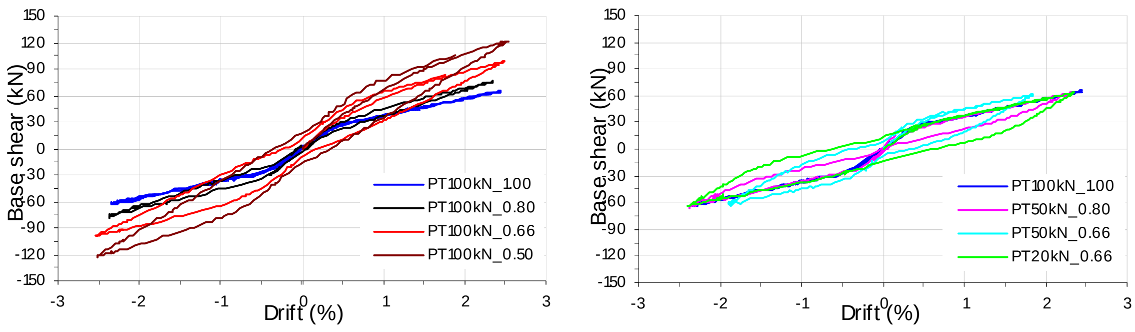


Figure 4.1. Numerical Results of Cyclic Testing for All Cases Considered

The Figure 4.1 (right) shows the cases in which the total design moment (M_t) was selected to be that of the PT100kN_1.00 case at 2.5% of drift and the values of β and initial post-tensioning were altered (i.e. overall resistance remained constant). As can be seen in the figure, this meant that all the design systems displayed significant hysteretic damping under the imposed cyclic loading as a function of the value of β selected.

4.2.Impulse Analysis

The second series of analyses performed involved the application of an impulse acceleration in order to study the free vibration of each system. Figure 4.2 shows the results for these analyses.
From Figure 4.2 (left) the effect of the addition of the dissipative steel angles on both damping and period is seen clearly. It can be noted however that only a slight difference exists between the damping of the system once the dissipative devices have been introduced. From Figure 4.2 (right) the effect of decreasing of the post tensioning force and the addition of the dissipative steel angles on both damping and period is seen clearly again.

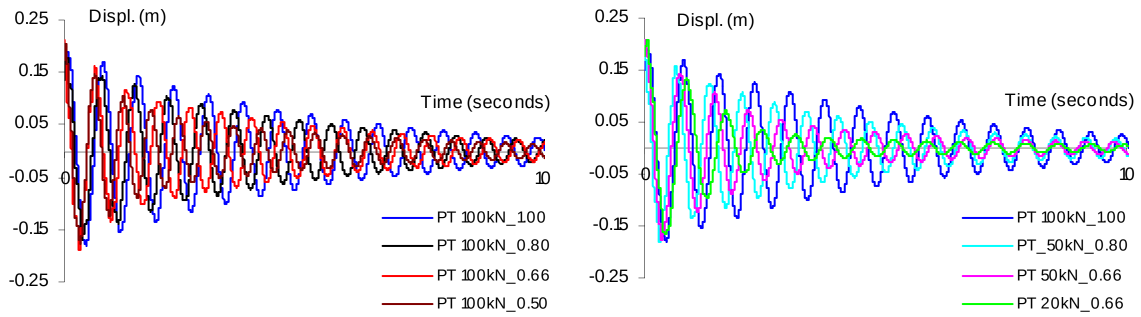


Figure 4.2. Numerical Results of Impulse Tests for All Cases Considered

4.3. Time History Analysis

In order to define the total system dynamic behaviour, several Non-linear Time History Analyses (NTHA) have being carried out. The performed analyses refer to a seismic input belonging to the set of Zone 1, Soil type B (named 000196xa), whose response spectrum was very close to the design response spectrum with PGA = 0.44g (see Figure 2.3).
The elaboration of the numerical outcomes emphasises the differences of seismic behaviour between the different model configurations, without and with the addition of steel elements. The detailed numerical seismic response of the experimental model in the reference configurations (see Table 4.1) Post-tension Only Case (PT100_1.00), and of two hybrid cases with the addition of steel elements (PT100_0.66) and with reduction of the PT force (PT50_0.80) are compared in Figure 4.3. The figure shows time history of (i) interstorey drift and (ii) accelerations at each floor.
As can be seen, the interstorey drift of the structure reduces in amplitude when additional steel devices are introduced. The values of interstorey drift at each storey are comparable for all cases. The presence of the steel dissipative angles allow a consistent reduction of the interstorey drift in both cases with lower values of maximum accelerations when the PT force was decreased.
Finally, the values of the (i) maximum interstory drift (MID), (ii) maximum acceleration (MA) and (iii) maximum base shear (MBS) obtained by NTHA considering all configurations are shown in Figure 4.4.
The presence of the steel dissipative angles allow a maximum reduction of the interstorey drift in the order of 1.5-2 times in comparison with PT only configurations. For the Accelerogram 000196x, it can be seen that: i) the MID decrease with growth in the dissipative capacity for β = 0.80 and 0.66, beyond this values starts to increase for β = 0.50, ii) minimum MA is obtained for β values equal to 0.66 and iii) MBS increases when the β values decrease. Varying the characteristics of the steel dissipative angles and the force in PT cables, the design procedure allows for the determination of equivalent configuration cases.

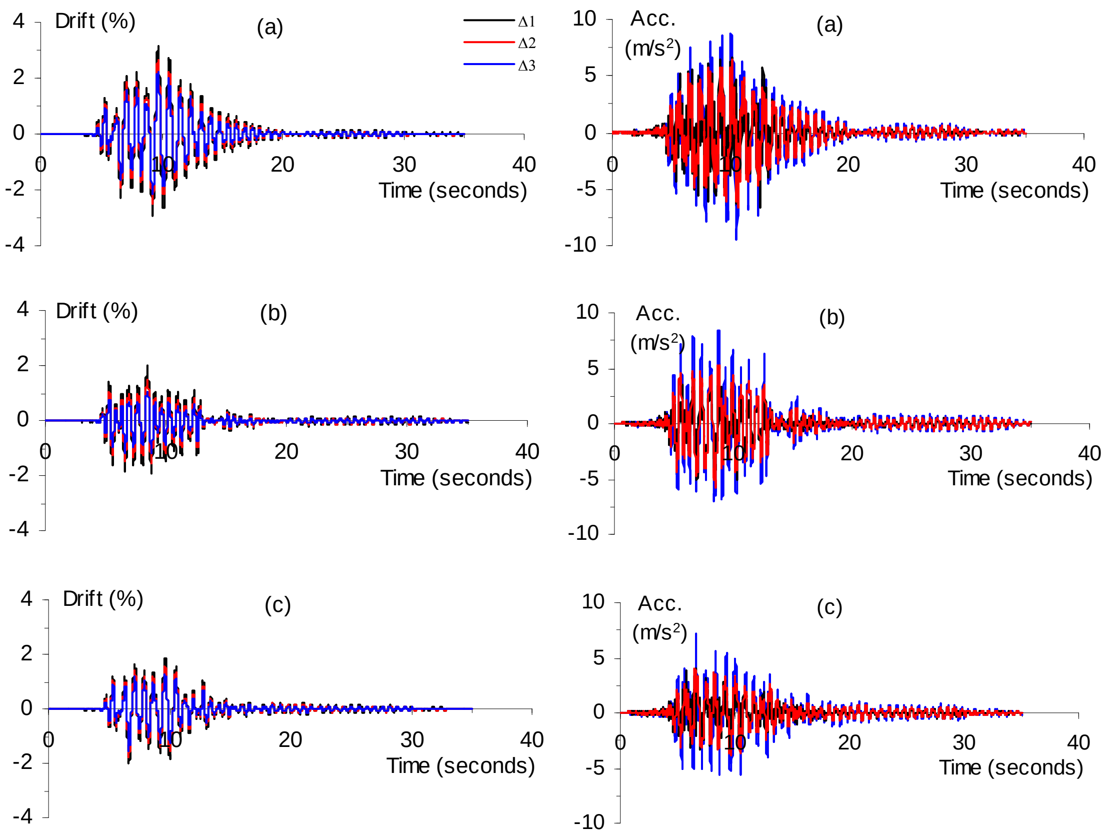


Figure 4.3 Time-Histories of Interstorey Drifts and Storey Acceleration (a) PT100_1.00, (b) PT100_0.66, (c) PT50_0.80 for Accelerogram 000196x.

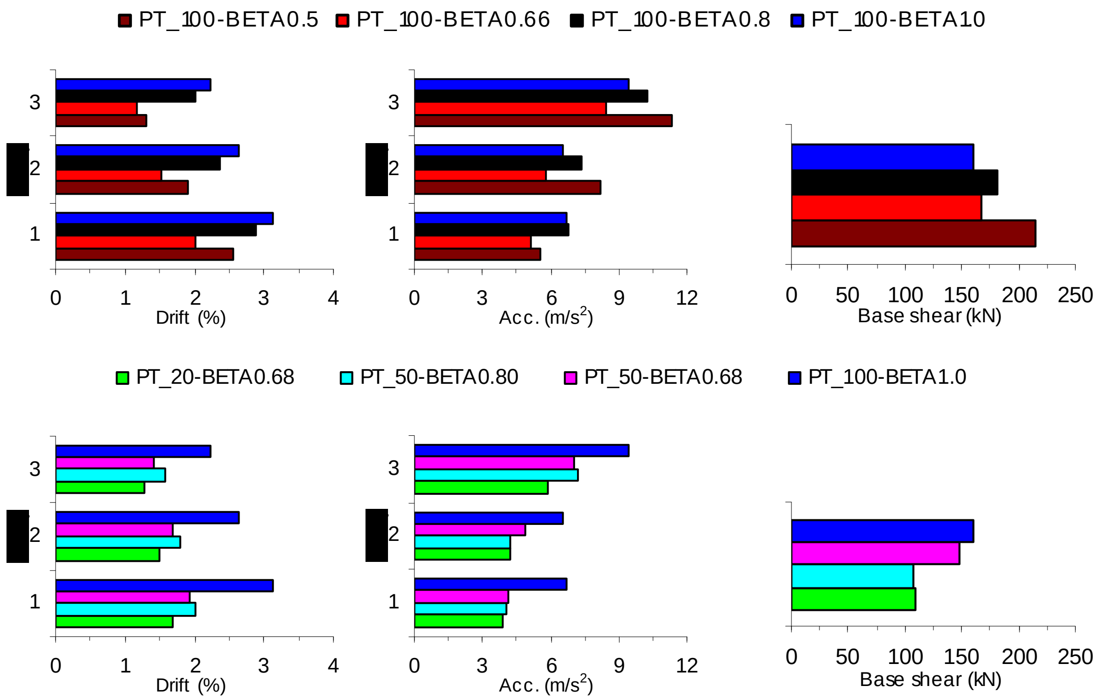


Figure 4.4. Numerical Results of NTHA: Comparison Between the Maximum Interstorey Drift, Story Acceleration and Base Shear (Accelerogram 000196x) Experienced by the Frame for All Cases Considered.

5. CONCLUSIONS

Non-linear analyses have been performed in order to define the test cases which will be used during the dynamic experimental tests on a 2/3rd scale, 3-dimensional, 3-storey, post-tensioned timber structure that will be performed in the Structural Laboratory of the UNIBAS, Italy, in collaboration with the UoC, New Zealand. Cyclic, impulse and time history analyses were performed in order to define total system behaviour and damping characteristics of the considered configurations. The numerical model has been considered with and without the addition of dissipative steel angles which are designed to yield at a certain level drifts with constant PT and varying PT. Cyclic and impulsive analysis have shown that all the configurations display significant hysteretic damping as a function of both PT force and factor β selected. NTHA results proved the effectiveness of dissipative steel devices in reducing seismic effects when compared to PT only cases. By varying post-tensioning forces and the contribution of steel dissipation devices in beam-column joints, a designer can control the global seismic response of the system. The configurations which will be used during the dynamic experimental tests have been identified. However further non-linear dynamic analysis are required in order to study the influence of global stiffness and strength of the structure on the seismic response under the complete set of natural earthquakes.

ACKNOWLEDGEMENT

The authors would like to acknowledge funding from the Structural Timber Innovation Company (STIC) and FederlegnoArredo (FLA). The work performed by G. Luongo on the numerical modelling is also acknowledged.

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