Proceedings of the Tenth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Pacific
6-8 November 2015, Sydney, Australia

Dynamic testing of a three-storey Pres-Lam Frame

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ABSTRACT: The following paper describes the second phase of dynamic testing of a three story Pres-Lam timber frame. This method of timber construction combines the use of large engineered timber members with post-tensioning cables/bars and is currently adopted throughout New Zealand and around the world. Pres-Lam structures withstand high levels of seismic loading without damage to the structural system.

The hybrid version of the Pres-Lam system improves seismic performance through the addition of external steel reinforcing. While the post-tensioning provides desirable recentering properties, the reinforcing devices dissipate energy from the system and increase moment resistance. During this phase, a second round of dynamic shaking table testing was performed on a 3-storey frame structure with improved detailing. The specimen was subjected to a selection of natural earthquake records with increasing (as a % of PGA) levels of seismic loading. The experimental model was tested with and without the use of dissipative angle devices. This paper presents the results of the shaking table testing looking at some key performance parameters. It shows how the addition of dissipative elements was successful in reducing drift without increasing base shear and consequent member forces.

1 INTRODUCTION

The Pres-Lam system uses engineered wood and post-tensioned elements in order to provide moment connections in timber structures. Adapting concepts and principles originally developed for precast concrete construction (Priestley 1991), this technique has been developed and extensively tested at the University of Canterbury (UoC) (Palermo et al. 2005). Although most experimental applications have used Laminated Veneer Lumber, Pres-Lam has also been tested using Cross Laminated Timber (Dunbar et al. 2014) and Glue Laminated structural members (Smith et al. 2014).

In medium and high seismic hazard areas the Pres-Lam system can combine the use of unbonded post-tensioned tendons with any form of dissipation device. While the post-tensioning provides desirable recentering properties, the dissipation devices allow adequate energy release as well as increasing moment resistance. During lateral movement, controlled rocking occurs which gives a “flag-shaped” hysteretic behaviour as shown in Figure 1. When dissipation devices are placed externally they become replaceable so that hysteretic energy dissipation is effectively decoupled from permanent structural damage. This decoupling leads to structures that will not only remain operational after a major earthquake event, but will also limit direct costs associated with repair and indirect financial losses associated with business operation and downtime.

An extensive dynamic experimental testing programme has been 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 seismic performance of the system focusing on the use of Glue Laminated structural members and the study of the systems real-time dynamic behaviour. During this phase, a second set of shaking table testing on a 3-dimensional, 3-storey timber frame structure has been performed at UNIBAS laboratory results of which are shown in this paper. As with the Phase One of this stage of testing (Ponzo et al.

2014), two values of the parameter β were investigated, with dissipative devices (β = 0.6) and without dissipative devices (β = 1.0), as described below.

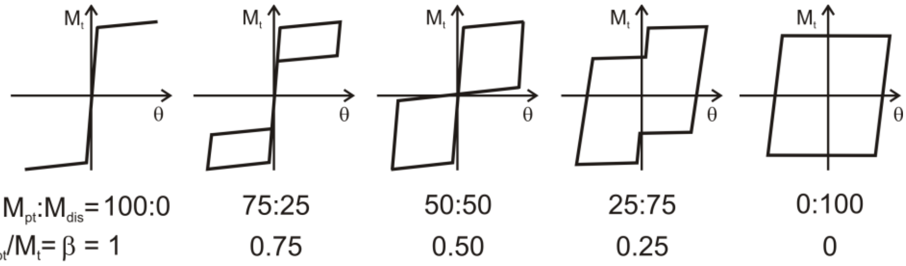

Figure 1. The Flag Shaped Hysteretic Loop

This paper will first briefly describe the detailing and testing set-up of the experimental model. Following this, testing results will be presented and studied in order to evaluate the impact that the design choices, especially the impact of the parameter β, has on the structural dynamic response.

2 THE STRUCTURE

The 2/3rd scaled test structure (Figure 2) is three stories and has a single bay in both directions. The test frame is made from Glulam (grade GL32h) and the building was designed to represent an office structure (live loading Q = 3kPa for level I and II) and has a roof garden (Q = 2kPa). The interstorey height of the structure is 2m and the building footprint is 4m x 3m. The post-tensioned bars, 26 mm in diameter, were stressed at 100kN. Dissipative 'Yielding steel angle' devices (Ponzo et al. 2011) were added to the structure in order to provide additional strength and reduce displacements without the increase of accelerations or base shears. These angles were added to the beam-column (only testing identified as 'with dissipation') and column-foundation (for both configurations tested) connections.

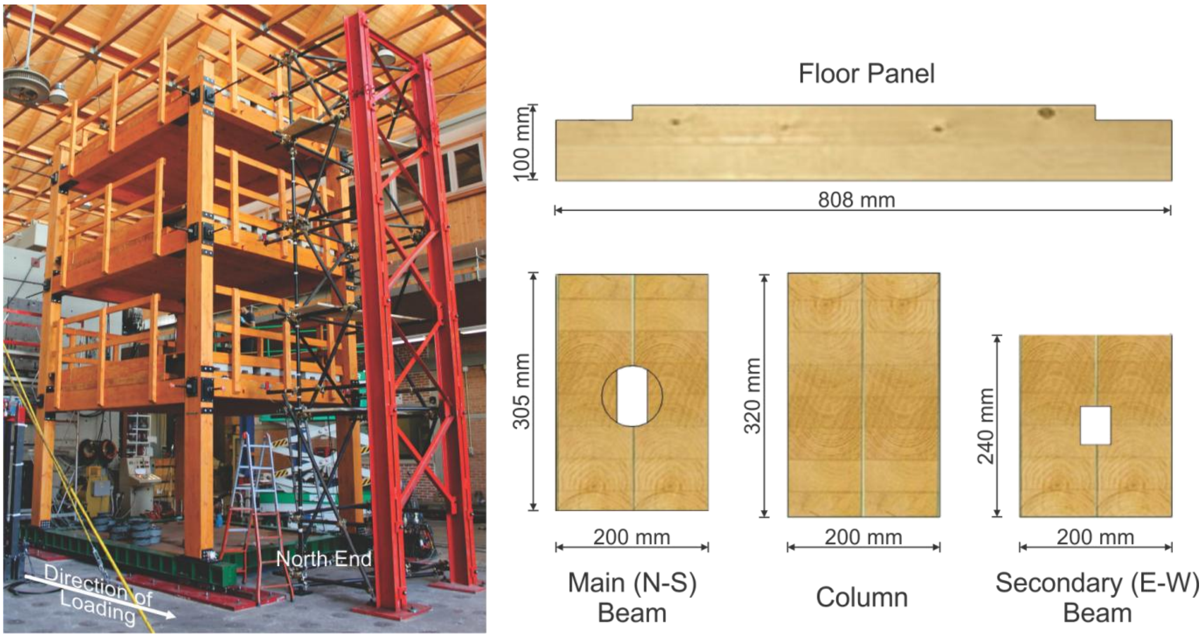

Figure 2: Test set-up for 2/3rd scaled, 3-storey dynamic frame specimen

During Phase Two alterations were made to the frame in order to improve the performance and contribution of the dissipative devices. Previously during Phase One testing, slip was noted between the base of the yielding angle and its attachment to the beam. Spring washers were added to the bolted connection in order to restrict this movement. Additional improvement of the Phase One frame configuration was made through the redesign of the column base connection. During Phase One testing, glued in rods were used in a form of steel shoe, which did not fully activate the steel angles due to the pull out failure of the glued in rods. Prior to Phase Two, four 10 mm thick plates were welded to the base plate and coach bolted (6 ϕ 12 mm, 80 mm long) to the side of the column.

2.1 **Instrumentation**

Instrumentation of the structure consisted of a combination of potentiometers, load cells and accelerometers. Fourteen horizontal accelerometers (two at each floor in each direction, one on the beam and one on the floor itself, and one in each direction on the shaking foundation) were placed on the structure. The storey displacements were measured directly with 2 potentiometers connected to each floor level and an external reference frame. A potentiometer was also attached to the shaking foundation.

Tension in the post-tensioning cables was measured 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 measured using a series of 3 potentiometers placed across the beam-column interface of one column on each floor and six potentiometers at the base of two columns. Base shear was measured directly using a load cell attached to the dynamic actuator. In total, 59 channels of data were used to record the real-time performance of the structure. This was assisted by various video and image equipment.

3 **DYNAMIC IDENTIFICATION OF TEST FRAME**

For each test case, dynamic identification testing by hammer impact was carried out in order to find the first three natural frequencies of vibration. In Figure 3 the Fast Fourier transforms used in order to identify the dynamic characteristics of the frame are shown with the experimental natural frequencies (f_i) corresponding to translational modes along the testing direction.

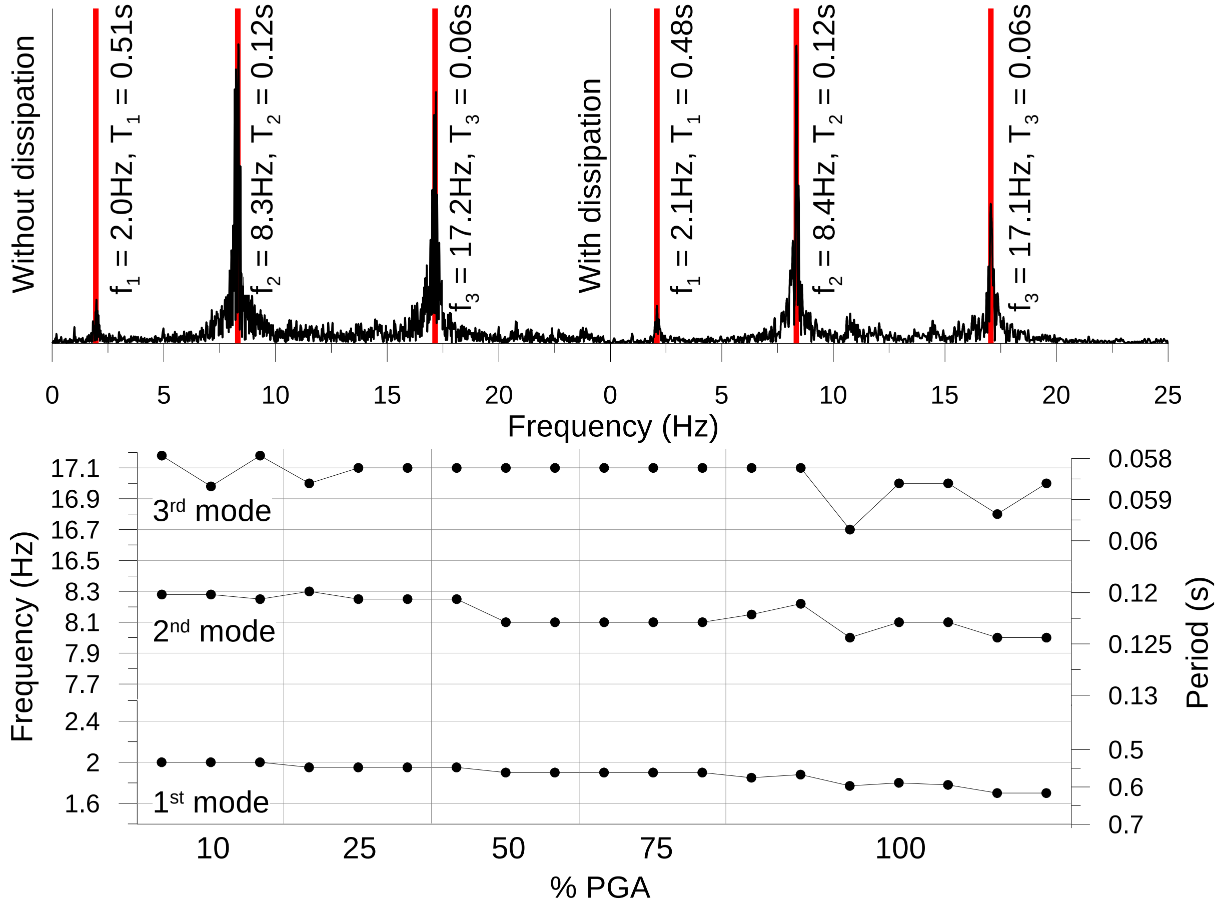

Figure 3: Fast Fourier transforms of dynamic test model 3rd floor acceleration response under hammer impact

Figure 3 displays a slight change in natural frequency between the test configurations with and without the dissipative reinforcing similar to that witnessed in Phase One of testing (Ponzo et al. 2014). As the mass remained effectively identical between the two tests this result implied a reduction in stiffness occurred between tests with and without dissipation. In Phase One of testing the first mode frequency of the specimen was 2.2Hz and 1.9Hz for testing with and without the angles, respectively. These frequency values are similar to the frequency values registered in Phase Two.

Figure 3 also shows the fundamental period of the structure calculated using the third floor accelerations recorded following the end of the strong motion of each test of the configuration with dissipation (each test is represented as a dot with three tests for each level of PGA%, and the full set of seven records at 100%, see Table 1). The figure clearly shows a decreasing trend in the fundamental frequencies of the dynamic test frame during the test sequence. This trend was also observed during Phase One testing with the final test leaving the specimen with a fundamental frequency of 1.9 Hz, reduced from 2.2 Hz. The fact that prior to Phase Two testing a fundamental frequency of 2.1 Hz was recorded indicates that these changes in frequency were not connected to accumulated structural damage but to the onset of slip in floor connections and other minor areas. It may also not be attributed to any losses in post-tensioning which remained almost constant over the entire test regime (in fact a 2% increase of post-tensioning was observed)

4 SHAKING TABLE APPARATUS AND SEISMIC INPUT

Testing was performed under dynamic loading in real time using a shaking foundation testing rig in the laboratory of the University of Basilicata. The steel foundation has a single degree of freedom in the N-S direction and is connected to an actuator that has a capacity of ± 500kN and a stroke of ± 250mm. The actuator is fixed to a hinge at the base of the foundation and pushes against a strong wall. The foundation is situated upon 4 SKF frictionless sliders 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 consisted of a set of 7 spectra compatible earthquakes selected from the European strong-motion database (Table 1). The code spectrum which was used when selecting the records was defined in accordance with the current Eurocode for seismic design (EN1998 2003) having a PGA of $a_g = 0.35$ and a soil factor of S = 1.25 (Soil class B – medium soil) giving a PGA of 0.4375g. Three of the earthquakes were further selected in order to provide a second, smaller group of test inputs (001228x, 000196x and 000535y). These records were selected as their average provided an adequate representation of the design spectra as shown in Figure 4.

During testing the full set of seven earthquakes was only used for the configuration with the dissipative angles when testing at 100% PGA. Testing at 10, 25, 50 and 75 % was performed with the smaller set of three earthquakes. The Erzican earthquake was not used at 100% of PGA for the configuration without dissipative angles due to the prediction of excessive drifts.

Table 1. Earthquake Characteristics

ID Code	Location	Date	M_W	PGA (g)	PGV (ms^{-1})	Epicentral Dist. (km)	Scale factor
001228x	Izmit, Turkey	17/08/99	7.6	0.357	0.332	47	1.5
000196x	Montenegro, Serbia	15/04/79	6.9	0.454	0.388	25	1.0
000535y	Erzican, Turkey	13/03/92	6.6	0.769	1.077	13	1.5
000187x	Tabas, Iran	16/09/78	7.3	0.926	0.844	57	1.0
000291y	Campano Lucano, It	23/11/80	6.9	0.264	0.413	16	1.5
004673y	South Iceland	17/06/00	6.5	0.716	0.720	15	1.5
004677y	South Iceland	17/06/00	6.5	0.227	0.208	21	1.0

Knowing the fundamental frequency of the structure as described above, 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 the earthquake demand at the fundamental period of the structure was on average higher than design.

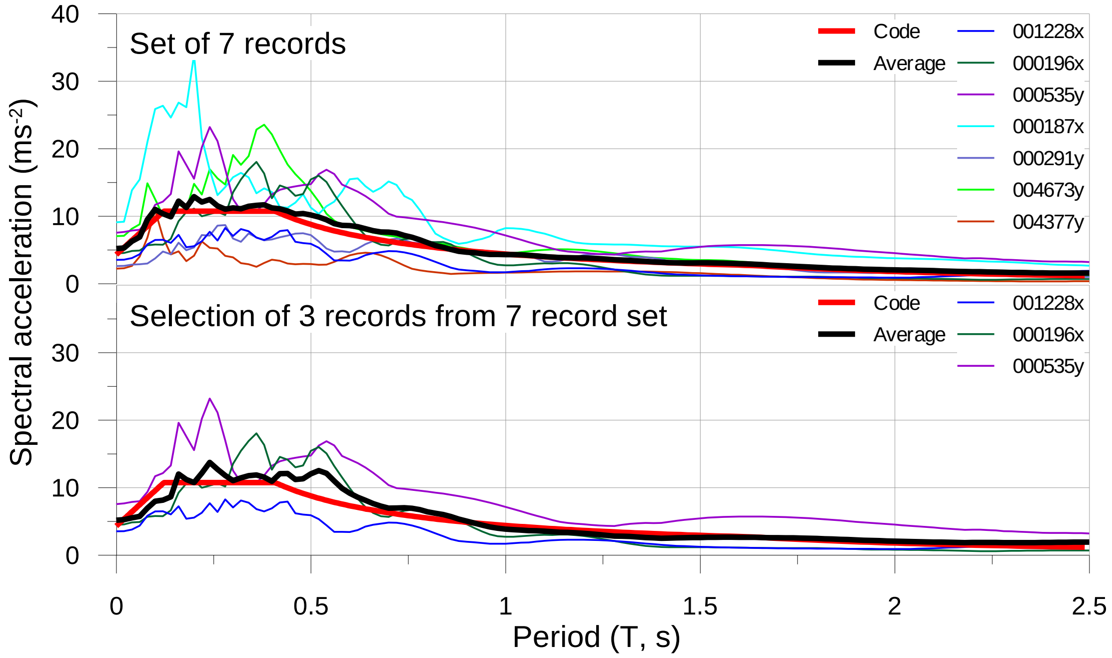

Figure 4: Selected earthquake input for shaking table testing

5 SHAKING TABLE TESTING RESULTS

The structure was first tested with the addition of the dissipative angles at beam-column joints and, upon completion, the angles were removed and testing without dissipation was performed. Figure 8 shows the ram force versus first floor drift for both test series subjected to 001228x, 000196x and 000535y at PGA75%. The figure shows the presence of the flag shaped hysteretic loop when the dissipative angles were added. The additional damping and strength led to a decrease in first floor drift across all testing while an increase in base shear was only observed during the 000535y testing.

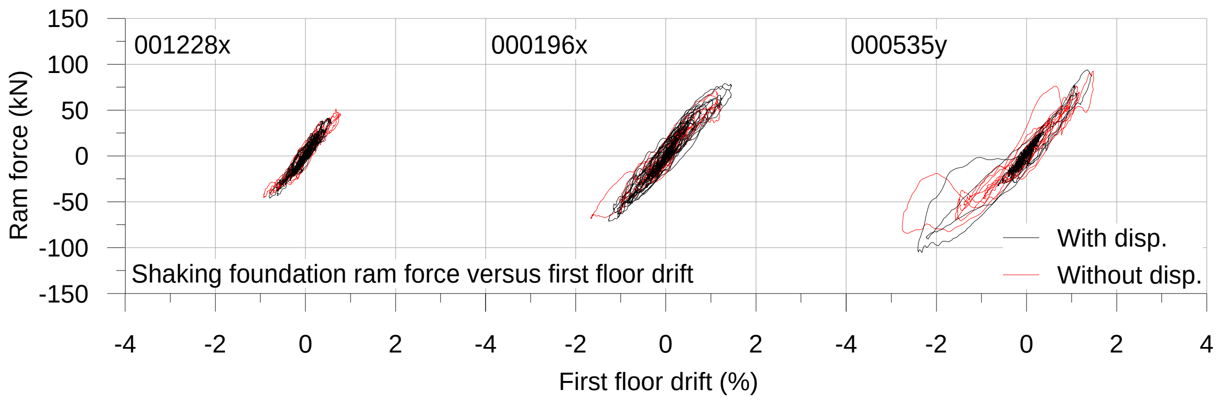

Figure 5: Shaking table force vs first floor drift for the model with and without dissipation (PGA75%).

Figure 6 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 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 indicating that the presence of dissipative reinforcing will not impact on serviceability level response.

Before gap opening has occurred the dissipative angle reinforcing remains nominally loaded. Following the PGA50% intensity level the response of the frame differed with a rapid increase of drift levels in the configuration without dissipation while for the dissipative configuration this rapid increase occurred following PGA75% testing. The presence of the steel dissipative angles led to an 18% decrease in average first floor drift between testing with and without the dissipative angles at

PGA75%. This was less than during Phase One of testing which displayed a 32% decrease at the same drift level. This was due to the impact of the improved base connection which created a more significant decrease in testing without dissipation. Between Phase 1 and Phase 2, there was a 22% decrease in average drift for the system without the dissipative angles at the beam-column joint compared with an 8% decrease when they were used.

Figure 7 shows the average maximum third floor acceleration for the two test configurations with increasing percentages of PGA. The figure shows that there was only a minor difference in third floor acceleration throughout testing. This included the periods during which the inter-story drifts were decreased due to the addition of the dissipative angles displaying their ability to reduce drift without increasing acceleration and force. Phase One of testing showed that without additional dissipation, accelerations in this type of structure are elevated which led to the conclusion that even in low seismic areas a minimum amount of dissipation should be applied (Ponzo et al. 2014). Phase Two testing has shown that the addition of very minimal dissipation ($\beta < 0.95$), in this case only to the base of the structure, is adequate to ensure that excessive accelerations do not occur.

The base shear response of the structure with and without dissipative reinforcing is also shown in Figure 7. As expected the base shear displayed the same general trend as the accelerations presented above. A maximum average value of 97kN was recorded for the configuration with dissipation corresponding to 100% of PGA intensity which was similar to the design level evaluated through the Pres-Lam design procedure (Smith 2014)

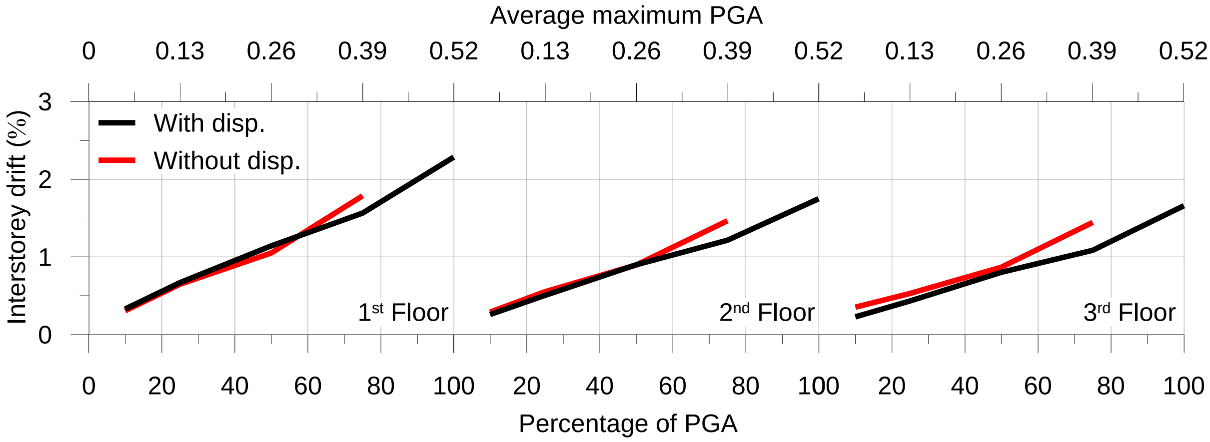

Figure 6: Comparison of maximum average drifts for test frame increasing PGA levels.

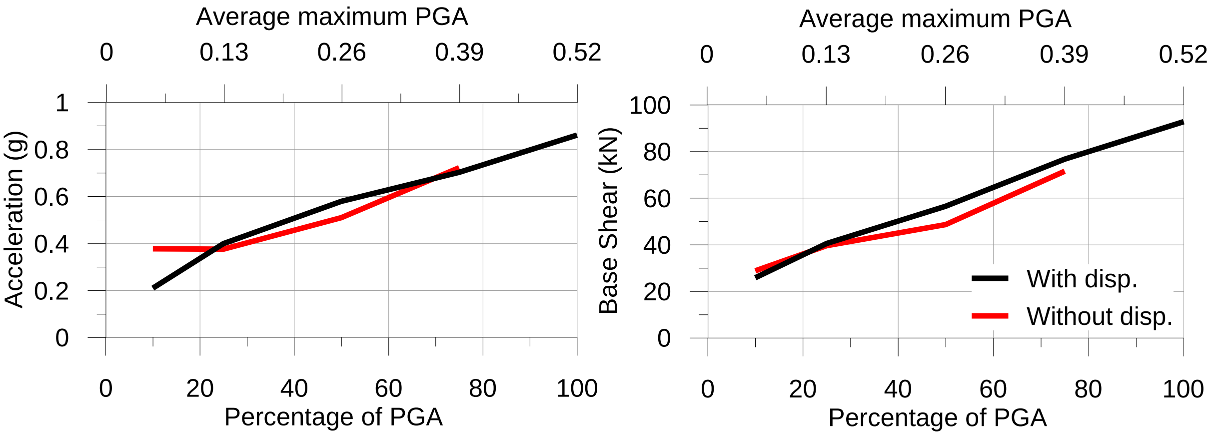

Figure 7: Comparisons of maximum average accelerations of test frame at increasing levels of PGA

Following the running of Phase Two, visual inspection of the test frame showed that the angles at the base of the columns had fractured. This was audible during the final test performed of the phase (000187 at 100% of PGA intensity). Although the fracture appeared to be due to fatigue phenomena, it occurred following the application of the full test sequence, a total of 80 earthquakes for Phase One and Phase Two, displaying significant resilience. These angles are able to be replaced following a major event if fracture occurs. Only minor damage to the Pres-Lam frame was noted following testing in the form of tension cracking in the first floor beams which did not impact on the performance of the structure. Fully threaded screws were used to reinforce this area prior the next phase of testing.

Figure 8: a) Fracture of dissipative angles at base and b) tension cracking of beam at first floor

6 THE RESPONCE OF THE PRES-LAM SYSTEM TO SEISMIC LOADING

A very stiff building designed for only elastic response will be subjected to very high accelerations and therefore high seismic forces. A Pres-Lam structural system impacts on the expected level of acceleration in two ways: through displacement ductility, and, when dissipative devices are attached, hysteretic damping.

The reason for the reduction in drift with no subsequent increase in base shear seen during testing is illustrated in Figure 9 where the seismic demand is represented as an Acceleration Displacement Response Spectrum (ADRS) (Chopra and Goel 1999). On the left of the figure the response of an elastic system and of the Pres-Lam frame without the additional dissipation are shown. The base shear of the structure is already significantly reduced from that of an elastic system by the non-linearity created by the gap opening. This means that although all materials in the structure remain elastic, the structure itself does not perform as an elastic structure.

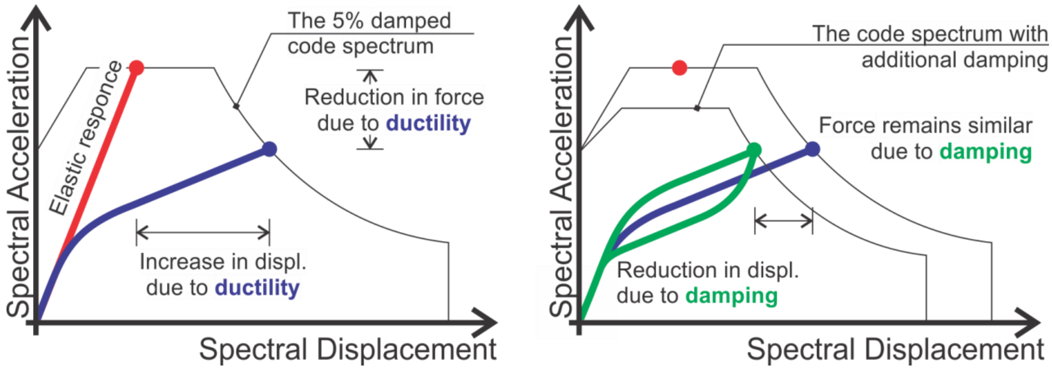

Figure 9: Effects of ductility and damping on the lateral load response of a structure.

On the right of Figure 9 the effect of adding the additional dissipation is shown and a stronger system with increased base shear and decreased drift is created. However this is accompanied by an increase in the damping capability of the system with lower seismic demand. This creates the situation where a decrease in displacement is observed without an increase in base shear.

One further impact of the addition of dissipation is the increase of the yield point of the system. Although the point of gap-opening is a function only of the initial post-tensioning amount across the interface, in a system with dissipation the joint also has the stiffness and strength provided by the angles following gap-opening. This leads to later onset of non-linear behaviour as shown in Figure 5 and evident in Figure 6.

7 CONCLUSIONS

Phase Two of an extensive dynamic test campaign has been performed on a Pres-Lam timber building in the laboratory of the University of Basilicata in Potenza, Italy. As during the previous Phase One of the project, the experimental model was tested both with and without the addition of steel angles at the beam-column interface which are designed to yield at a certain level of drift.

This paper has briefly described the detailing and testing set-up of the experimental model including a description of the column-foundation connection which was improved following Phase One of testing. Test results have shown how providing capacity with dissipative reinforcing can reduce displacement without increasing acceleration (and therefore base shear). The addition of this reinforcement will not, however, impact on serviceable displacements which may govern section sizing.

Study of the frequency and period response of the structure before each of the test configurations has displayed a slight change in the fundamental frequencies of the frame which was shown to decrease during testing however had increased since the end of Phase One testing. By monitoring the amount of post-tensioning in the structure during testing it was shown that the changes in frequency were not connected to changes in post-tensioning force.

Explanation of the performance of a Pres-Lam frame was also made through the study of an ADRS spectrum. This showed that without the placement of dissipative devices almost all materials remain elastic however the system does not respond elastically. When dissipation is added, the reduction in drift comes from the increased strength of the joint and the additional damping which they provide to the system.

In the next Phase Three of the UNIBAS–UoC collaboration, the experimental model will be tested on the shaking table adding bracing systems in order to evaluate the effects of other forms of seismic energy dissipation on Pres-Lam structures and improve system serviceability performance.

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