FRAGILITY CURVES FOR POST-TENSIONED TIMBER FRAMES

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ABSTRACT

Since 2010, it has been observed a significant worldwide increment in the construction of post-tensioned timber (Pres-Lam) buildings. In high seismic areas, Pres-Lam technology combines unbonded post-tensioning tendons and supplemental damping devices to provide moment capacity to beam-column, wall-foundation or column-foundation connections. In low seismic areas, designers may choose to not provide additional damping, relying only on the post-tensioning contribution. Post-tensioning decreases as time passes due to creep phenomena arising in compressed timber members, a reduction of the joint moment capacity, and anticipated rocking motion triggering may occur. This paper investigates the seismic performance of a post-tensioned timber frame building designed with and without supplemental damping devices by fragility analysis. Specifically, fragility curves are computed accounting for the amount of post-tensioning loss expected in fifty years. Results show that in both cases the structural performance, measured by the interstorey drift and re-centering capability, is not significantly affected over time. Furthermore, if dissipaters are provided, they contribute to reducing the expected increase of interstorey drift due to post-tensioning losses.

Keywords: PresLam; Long-term; Post-tensioning loss; Fragility curves; Post-tensioned timber

1. INTRODUCTION

As a result of the Precast Seismic Structural System (PRESSS) program (Priestley 1991), the hybrid connection proved to be an efficient low-damage solution for precast concrete buildings. The innovative connection combines unbonded post-tensioning tendons and mild steel bars to accommodate the seismic demand through controlled rocking between structural elements. While tendons provide re-centering capability to the system, supplemental damping allows for hysteretic energy release as well as providing additional moment capacity.

In 2002, the concept was transferred to steel members (Christopoulos et al. 2002) providing the hybrid system being material independent. In 2005, the technology was extended to engineered timber products at the University of Canterbury (Palermo et al. 2005; Buchanan et al. 2008; Newcombe et al. 2008) and referred to as the Pres-Lam system. Since 2010, several PresLam buildings (Figure 1) have been built in New Zealand, e.g. (Brown et al. 2012), and overseas, e.g. (Leyder et al. 2015), including different lateral load resisting systems: post-tensioned timber walls (Sarti et al. 2015) and post-tensioned timber frames (Di Cesare et al. 2017).

It is widely accepted that post-tensioning loss deriving from creep phenomena in compressed timber members lead to a reduction of the connection moment capacity. Experimental and analytical studies (Davies and Fragiaco 2011; Fragiaco and Davies 2011; Wanninger et al. 2014; Granello et al. 2017) have shown that expected losses are in the range of 6-50% in 50 years in most practical cases, mainly depending on the ratio between post-tensioning steel and timber and on the amount of timber loaded perpendicular to the grain.

Numerical studies (Granello et al. 2018 (In Press)) showed that that losses up to 30-50% have little effect on the seismic performance of PresLam frames at the Ultimate Limit State. This is because the dissipaters (if properly designed) are able to absorb the extra amount of moment demand due to the

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post-tensioning loss without increasing the building’s interstorey drift. If additional damping is not provided, the interstorey drift can still be controlled; however, greater increase in terms of drift is expected. Results show that post-tensioning losses greater than 30-50% strongly affect the serviceability limit state of the system. Specifically, the rocking motion can be triggered by low-intensity earthquakes causing damage to no-structural elements.

Figure 1. Post-tensioned timber buildings: a) Massey University, Wellington, New Zealand (courtesy of Andy Buchanan; b) ETH House of Natural Resources, Zurich, Switzerland (copyrights Marco Carocari/ETH Zurich); c) Trimble Navigation Offices, Christchurch, New Zealand (courtesy of Paul Drummond); Merritt Building, Christchurch, New Zealand (courtesy of Andy Buchanan).

In this paper, the seismic response of the post-tensioned frame building analyzed in (Granello et al. 2018 (In Press)) is numerically investigated over time by means of fragility analysis.

2. CASE STUDY BUILDING

A case study building (Figure 2) was designed to evaluate the loss influence on the seismic performance. The building is a further development of the design example proposed in (Pampanin et al. 2013). The structure was designed to be located in Christchurch on a type D soil (maximum spectral acceleration 0.9g for a 500 years return event) and in Plymouth on a type D soil (maximum spectral acceleration 0.54 g for a 500 years return event). Columns and beams are made of Laminated Veneer Lumber (LVL) grade 16. Sections are made of 900 x 441 mm and 650 x 441 mm for columns and beams, respectively. Post-tensioning tendons were selected to be 7-wire strands, whereas ‘plug and play’ devices (Sarti et al. 2016) provide additional damping for the building in Christchurch (Figure 3). While post-tensioning tendons are positioned at the section centroid of the beam section, dissipaters are placed ± 275 from the beam centerline. Steel plates 20 mm thick were designed to protect the column perpendicular to grain at the rocking interface, avoiding timber crushing. They also have a beneficial effect reducing the amount of post-tensioning loss (Granello et al. 2017). Further details are reported in (Table 1).
Figure 2. Case study building: a) side view and b) plan view.

Figure 3 Beam-column joint detailing and dissipaters detailing.

Table 1. Post tensioning and dissipaters detailing.

<table>
<thead>
<tr>
<th>Storey</th>
<th>Tendons number</th>
<th>Post-tensioning [KN]</th>
<th>Tendons Stress [%f_{pt01k}]</th>
<th>Mild Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>With Dissipaters</td>
<td>1&amp;2</td>
<td>3</td>
<td>300</td>
<td>60%</td>
</tr>
<tr>
<td></td>
<td>3&amp;4</td>
<td>2</td>
<td>200</td>
<td>60%</td>
</tr>
<tr>
<td>Without Dissipaters</td>
<td>1&amp;2</td>
<td>2</td>
<td>200</td>
<td>60%</td>
</tr>
<tr>
<td></td>
<td>3&amp;4</td>
<td>2</td>
<td>200</td>
<td>60%</td>
</tr>
</tbody>
</table>
A semi-rigid moment connection (Figure 4) was designed at the column-foundation level for the building in high seismic zone by providing threaded steel bars embedded with epoxy. Analysis showed that despite the satisfactory performance for strong and rare events (e.g. return period equal to 500 years), drift limits (often controlled by non-structural damage) were exceeded in the case of less intense but more frequent (e.g. return period equal 25 years) earthquakes. Therefore, a stiff connection was necessary to limit the first-floor drift. The option of introducing external dissipaters, which are easier to replace, was also explored. However, this solution was unfeasible due to the high number of connectors necessary between the dissipaters and the column. Shear keys were also provided for transferring shear and avoiding the internal bars working in dowel action.

3. POST-TENSIONING FORCE OVER TIME

In Figure 5a the amount of post-tensioning force over time is reported for a PresLam frame building (Brown et al. 2012) under monitoring in Christchurch, New Zealand. The prediction according to the procedure proposed in (Granello et al. 2017) is also reported. It can be seen that the average post-tensioning trend can be predicted with reasonably accuracy within the observation period. However, the uncertainty of the prediction increases over time because the response of each load cell is subjected to intrinsic variability.

Figure 5b reports the standard deviation of the error (STD) between the experimental data and the model. As anticipated earlier, the standard deviation increases over time. The time evolution of the standard deviation was fitted with a power law, i.e. \( \text{STD} = at^b \), and extrapolated to 50 years. Figure 6 reports the post-tensioning trend for the case study building according to (Granello et al. 2017).
values plus/minus STD and plus/minus 2STD were calculated by considering the exponential law presented in Figure 5. An average post-tensioning loss equal to 16% is predicted in 50 years.

<table>
<thead>
<tr>
<th></th>
<th>0 years</th>
<th>5 years</th>
<th>10 years</th>
<th>25 years</th>
<th>50 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>$PT_{\text{avg}}$</td>
<td>100%</td>
<td>92%</td>
<td>91%</td>
<td>87%</td>
<td>84%</td>
</tr>
<tr>
<td>$PT_{\text{STD}}$</td>
<td>100%</td>
<td>89%</td>
<td>86%</td>
<td>79%</td>
<td>69%</td>
</tr>
<tr>
<td>$PT_{2\text{STD}}$</td>
<td>100%</td>
<td>87%</td>
<td>82%</td>
<td>70%</td>
<td>55%</td>
</tr>
</tbody>
</table>

Figure 6 Post-tensioning loss prediction for the sample building including uncertainty.

3. MODELLING

The moment-rotation behavior of a post-tensioned timber connection was defined using an iterative analytical procedure developed by Pampanin et al. (2001), modified by Palermo (2004), extended to the Pres-lam system by Newcombe et al. (2008), and further developed by Smith et al. (2014). This last approach showed a satisfactory (with limited error) prediction of the post-tensioned behavior of timber frames (Ponzo et al. 2017). Then, lumped plasticity models (see Figure 7) were calibrated against the moment-rotation response using rotational springs in parallel and in series as follow: (i) a multi-linear elastic hysteresis for the post-tensioning contribution, (ii) an elasto-plastic rule for the mild steel contribution and (iii) an elastic-rigid rule for the internal rotation before the gap opening contribution. An additional rotational spring was placed at the beam-column joint to take into account the joint shear stiffness (Smith et al. 2014).

4. PERFORMANCE LEVELS

The following performance levels were considered:

- **SLS1 or Serviceability Limit State 1**: it is reached if dissipaters are subjected to yielding. The dissipaters can be replaced after the event at moderate cost, especially if they are external thus easily accessible.
- **SLS2 or Serviceability Limit State 2**: It is reached if damage is expected to no-structural elements. This second case is assumed to occur when the interstorey drift is greater than 0.33% according to the New Zealand Standard 1170.0.
- **ULS or Ultimate Limit State**: It is reached if damage is expected to occur on the main structural elements (timber yielding at the rocking interface) or the interstorey drift is greater than 2.5%.
• CLS or Collapse Limit State: It is reached if excessive interstorey drift greater than 6% occurs or the system fails (e.g. yielding of the tendons or timber ultimate limit strain). Given the connection used in this study, the system failure occurs beyond 7% drift. Therefore, CLS is governed by interstorey drift limits.

• RLS or Reparability Limit State: It is reached if the residual drift after the earthquake is greater than 0.5%. In this scenario, the building is considered economically not repairable, that is the cost of reparation is higher than replacement.

Figure 2 Performance levels at the beam-column joint interface of the first/second floor.

5. FRAGILITY CURVES

The spectral acceleration is the intensity measure (IM) selected for fragility analysis. The procedure used for their computation is the commonly known “multi-stripe analysis method” (Baker 2015). The procedure is described in the following steps:

1. Subdivision of the intensity measure domain in stripes. In this case, each stripe was represented by the spectrum given by the New Zealand Standard 1170.5 for 20, 25, 50, 100, 250, 500, 1000, 2500 years return period, respectively.

2. Selection of N ground motions compatible with each stripe for each site. In this case 80 ground motions were selected for each stripe for each location i.e. Christchurch and New Plymouth. Due to space limits, only the ground motions selected for the events with return period equal to 25 and 500 years for Christchurch are reported in Figure 9a and b, respectively.

3. Execution of N NLTH analyses for each stripe. Extraction of the maxim interstory drift for each analysis. Classification of the structural response by the specified performance levels.

4. Definition of the fragility function via Maximum Likelihood Estimation (Baker 2015). In this study, a lognormal fragility function was used.

Figure 9 Ground motions selected with a) 25 years return period and b) 500 years return period.
The fragility curves at the initial time for the building with dissipaters are reported in Figure 10a. It can be noticed that the building has approximately 15 % probability of damaging the no structural elements (SLS2) for a seismic event with return period equal to 25 years, approximately 2% probability of damaging the structural elements (ULS) by an event with return period equal to 500 years and less than 13% probability of exceeding 6% drift (CLS) under an event with return period equal to 2500 years. Furthermore, for events with return period lower than 500 years the building shows a probability greater than 99.9% to have residual deformation smaller than 0.5% drift. Finally, there is approximately 75% probability that the dissipaters are subjected to yielding for an event with return period equal to 25 years.

Figure 10b reports the family of fragility curves at 50 years. The lower bound is represented by a scenario with post-tensioning loss equal to 45% (i.e. the expected average value minus 2 standard deviation). Results shows that the performance at SLS2, ULS, CLS and RLS is similar to the initial one. However, the probability of yielding the dissipaters increases from 75% to almost 90% for an event with 25 years return period.

This confirms what found in (Granello et al. 2018 (In Press)) : the dissipaters are earlier activated with post-tensioning loss because of the clamping force between the beam and the column is reduced. Therefore, they start dissipating energy at lower level of drift. Because of this, the interstorey drift does not significantly increase although the connection capacity is reduced.

However, they are activated more often during the building life, as event with lower return period can easily trigger the rocking motion. If dissipaters are external (e.g. the beam-column joint case), the substitution cost is essentially related to the cost of the material itself and therefore not considerably expensive (if compared to the no structural elements ad example). However, if dissipaters are internal (e.g. column-foundation case) the replacement might be more complicated with consequent higher cost.

Figure 11a reports the fragility curves for the building without dissipaters at initial time. It can be noticed the specimen shows approximately 1% probability of damaging the no-structural elements for an event with 25 years return period, approximately 5% probability of damaging the structural elements for an event with 500 years return period and less than 5% probability of overcoming 6% drift for an event with 2500 years return period. Furthermore, the building shows more than 99.9% probability of having a residual interstorey drift lower than 0.5% for all the event with return period below 2500 years.
It can be noticed from Figure 11b that the area enclosed between the SLS, ULS and CLS curves at initial time and the same curves at 50 years, is greater respect to the case of the building with additional damping. This means that post-tensioning losses have a greater impact when no dissipaters are provided, and the consequent shift of the fragility curve at 50 years is higher (with respect to the building with additional dissipaters). When looking at design code provisions, the probability of exceeding the SLS, ULS and CLS limit state for specific events with 25, 500 and 2500 years return period, rises approximately to 7%, 7% and 8%, respectively. This means that the building still shows acceptable code compliant behaviour after 50 years. However, from the pure seismic performance point of view, the greater shift in the fragility curves over time proves that dissipaters mitigate the effect of post-tensioning loss in terms of overall damage.

In terms of re-centring, the building with no dissipaters after 50 years still maintains a probability of exceeding the RLS lower than 0.1% an event with return period lower than 2500 years. Since dissipaters are not provided, the only post-tensioned joint is able to re-center although losses occur.

5. CONCLUSIONS

The paper presented a fragility analysis as function of time for two PresLam frame buildings designed respectively in Christchurch (high seismic hazard) and New Plymouth (low seismic hazard), New Zealand. The building in Christchurch was designed by combining unbonded post-tensioned tendons with dissipaters, while the building in New Plymouth relies only on unbonded post-tensioned tendons. A set of performance levels was described for post-tensioned timber rocking structures. The post-tensioning force over time was predicted by using an available methodology, and the uncertainty in the prediction was also considered. Results show that both structures (with and without dissipaters) provide a satisfactory performance while considering damage to no structural elements, damage to structural elements and excessive interstorey drift for events with 25 years, 500 years and 2500 years return period, respectively. Furthermore, both buildings show high probability of re-centering, i.e. greater than 99.9%, for events with return periods lower than 500 years. If dissipaters are provided, they contribute to reducing the expected increase of interstorey drift due to post-tensioning losses over time.

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7. REFERENCES


