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PERFORMANCE BASED DESIGN FOR POST-TENSIONED TIMBER FRAMES

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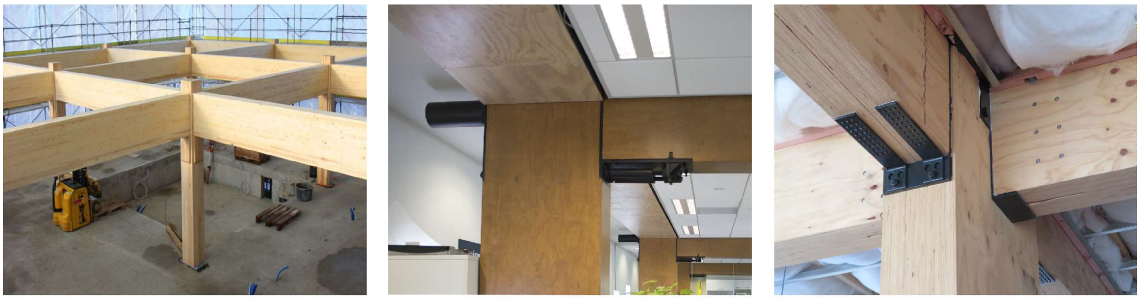
Abstract: *Pres-Lam technology combines unbonded post-tensioning tendons and supplemental damping devices to provide moment capacity to beam-column, wall-foundation or column foundation connections. the concept consists of accommodating the seismic demand through controlled rocking mechanism, while dissipating energy in specific replaceable devices.*

The seismic design of PresLam structures is normally carried out by using Force-Based Design (FBD) or Displacement Based Design (DDBD) procedures. Normally the DDBD approach is preferred, as the structure can be designed to achieve a set of target displacement limits. However, in both FBD and DDBD procedures the design of structural elements is normally carried out by considering the ultimate limit state (ULS), and subsequently check the serviceability limit states (SLS). Experimental and numerical results have shown that post-tensioned timber frames present high elastic deformation before the rocking motion activation; therefore structural members size is often governed by SLS. A Performance Based Design (PBD) procedure is herein proposed for the design of post-tensioned timber frames. Such procedure is applied to the design of a four-storey Pres-Lam frame, and non-linear pushover analyses are used to verify the accuracy of the methodology proposed.

Introduction

High performance multi-storey timber frame system incorporating Pres-Lam technology has been developed at the University of Canterbury (Palermo *et al.*, 2005). Pres-Lam technology uses post-tensioning tendons or bars passing through internal ducts in large timber box beams, frames or walls. In moment-resisting timber frames, the horizontal steel tendons in the beams also pass through the columns, providing moment resistance. Internal or external dissipaters can be added to provide supplemental damping to the system. Post-tensioned timber is suitable for a wide range of building types, including commercial structures, and has the potential to compete with existing forms of construction in terms of cost, versatility and structural performance (Smith *et al.*, 2009).



a) b) c)

Figure 1. Post-tensioned timber buildings. (a) ETH House of Natural Resources (HoNR), Zurich (copyright of ETH Zurich/Marco Carocari). (b) Merritt Building, Christchurch; (c) Beatrice Tinsley, Christchurch.

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The connection technique (Palermo *et al.*, 2005) was adapted from post-tensioned pre-cast concrete systems (Priestley *et al.*, 1999; Pampanin, 2005). The basic concept is that seismic movements are accommodated through a controlled rocking mechanism between prefabricated elements, developing elastic elongation of long lengths of unbonded high-strength steel tendons, with energy dissipation provided by the yielding of replaceable steel devices. Figure 2a shows an internal beam column joint in a timber frame. Figure 2b shows the typical flag-shaped moment-rotation response of a hybrid external beam-column joint with elastic post-tensioning and yielding steel dissipaters.

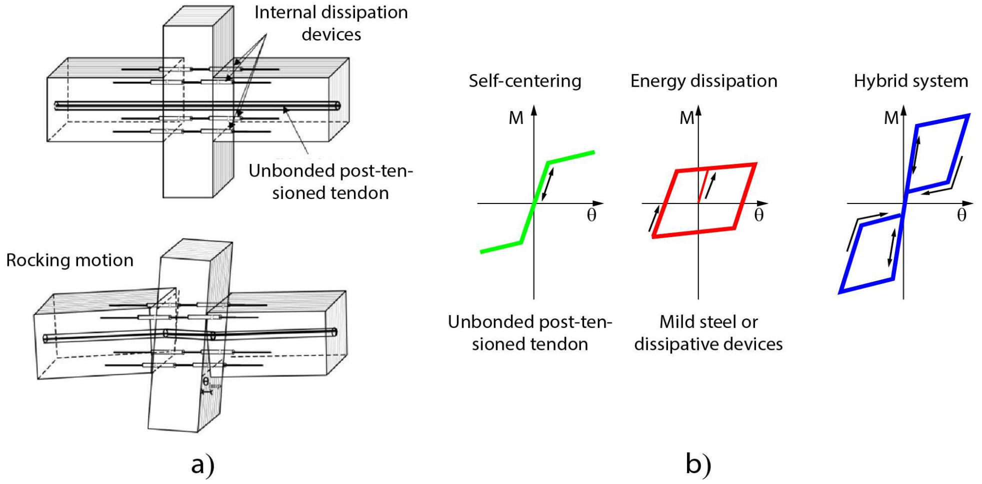


Figure 2. a) Internal beam column joint in a timber frame and b) flag-shaped moment-rotation response of a hybrid connection (modified from Palermo et al., 2005).

One significant difference between seismic design of structural timber buildings and reinforced concrete buildings is the difference in elastic deformations between the two materials. A multi-storey building is likely to have similar member sizes, whether designed in timber or concrete, because the reduced mass of the timber building is roughly offset by the lower modulus of elasticity of wood ($E_{wood} \cong 1/3\ E_{concrete}$). Hence the timber building will have larger elastic deformations under the design level earthquake forces, so that the gap opening at the rocking interfaces will occur later, followed by activation of any yielding devices for energy dissipation. Therefore, the serviceability limit state lateral load design usually governs the size of the members and the amount of post-tensioning, especially if stringent displacement limitations, according to current code provisions, are imposed to protect non-structural partitions, ceilings and facades.

In spite of that, in the common practice procedure for the design of post-tensioned timber frames the structure is designed at ultimate limit state and the serviceability limit state is subsequently checked. With respect to a performance-based design methodology, this paper describes a simplified design procedure that allows the determination of the required section sizes and post-tensioning forces to achieve the lateral strength demand of the frame, without exceeding the total allowable displacements for either serviceability or ultimate limit states.

Procedure description

As introduced, in the common practice procedure for the design of PresLam structures the performance objective at ultimate limit state (ULS) and at serviceability limit state (SLS) is guaranteed by designing at ULS and checking at SLS. According to the fact that the size of the members and the amount of post-tensioning are usually governed by SLS, a Performance Based Design (PBD) procedure is herein proposed to directly control the performance levels both at SLS and ULS.

PBD procedure is carried out by dimensioning section sizes and post-tensioning so that:

- the elastic connection rotation θ_y is less than the imposed limit drift at SLS θ_{SLS};
- the moment capacity of the connection at the imposed SLS drift M_y is more than the connection moment demand obtained by DDBD procedure M_{SLS};

The moment capacity at ULS M_u is then checked and mild steel reinforcement is added to ensure enough redundancy of the structure and, when necessary, to increase the moment capacity at ULS.

The performance design objective and the generic design procedure for post-tensioned timber frame are summarized in figure 3 and 4.

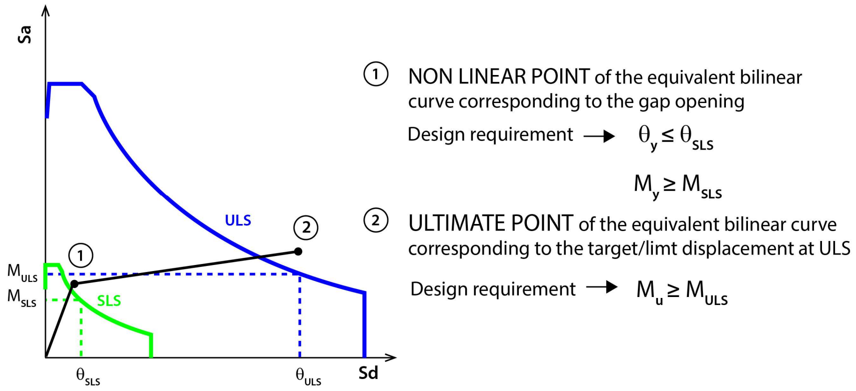


Figure 3. Performance Base Design procedure for post-tensioned frames, design objective.

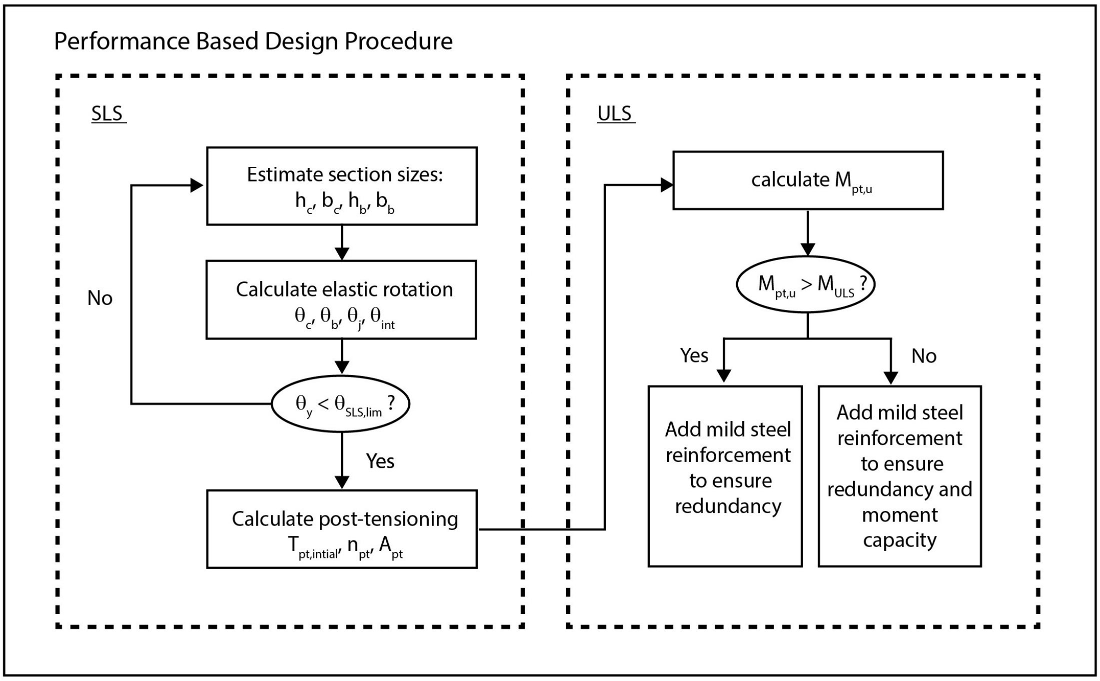


Figure 4. Flow chart of Performance Based design procedure for post-tensioned timber frame.

Step 1. Demand evaluation at SLS and ULS: Displacement Based Design

The proposed approach uses the common direct displacement based design (DDBD) to obtain the seismic base shear corresponding to the displacement limit of both SLS and ULS. DDBD has been developed as a simple method for designing to achieve a set displacement limit. The procedure involves characterizing the structure by an effective stiffness to the design displacement and a level of equivalent elastic damping, which combines the effects of elastic and hysteretic damping. The general procedure for DDBD is shown in Figure 5. A more detailed description of the method can be found in many other references in literature and thus will not be repeated here (Priestley *et al*,. 2007).

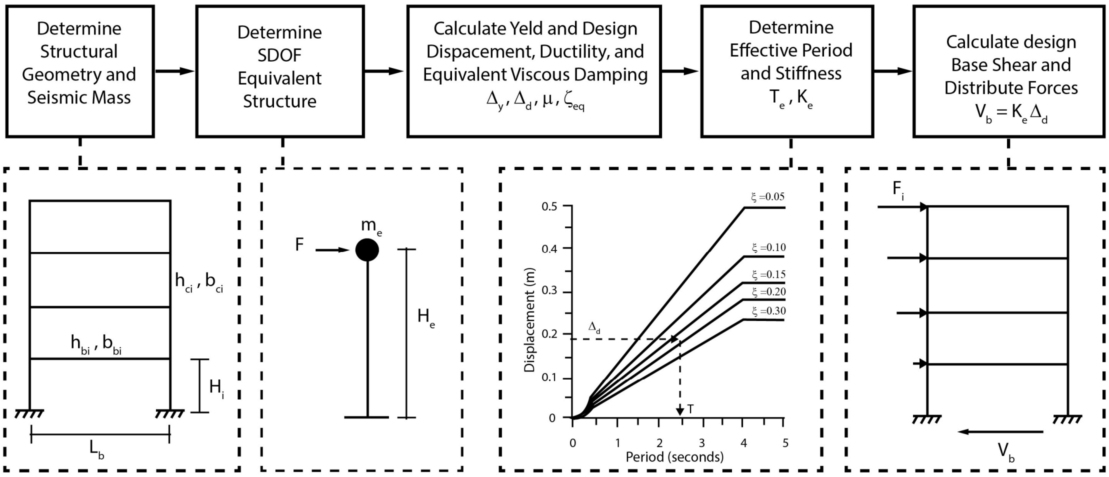


Figure 5. Direct Displacement Based Design steps (modified from Priestley, 1998 - Priestley et al., 2007 and Sporn and Pampanin, 2013).

The seismic base shear is distributed to the structure using the equivalent static analysis defined by New Zealand standards (NZS 1170.5, 2004) and the internal forces inside the frame are distributed using the equilibrium method proposed by (Priestley *et al*,. 2007).

Step 2. Beam and column dimensioning at SLS

As for a traditional elastic timber frame, the initial estimation of beam and column dimensions is based on gravity loads. Member size are then confirmed or modified in order to satisfy the inter-storey drift limit at SLS.

The deformation of a post-tensioned timber frame is made up of several contributing displacement sources:

- The elastic beam deformation θ_b
- The elastic column deformation θ_c
- The elastic joint deformation θ_j
- The connection deformation θ_{con}. The connection deformation is made of two separate contributions: the interface compression deformation θ_{int} and the gap deformation θ_{gap} due to the gap opening beyond the elastic deformation.

A summary of rotations is shown in figure 6:

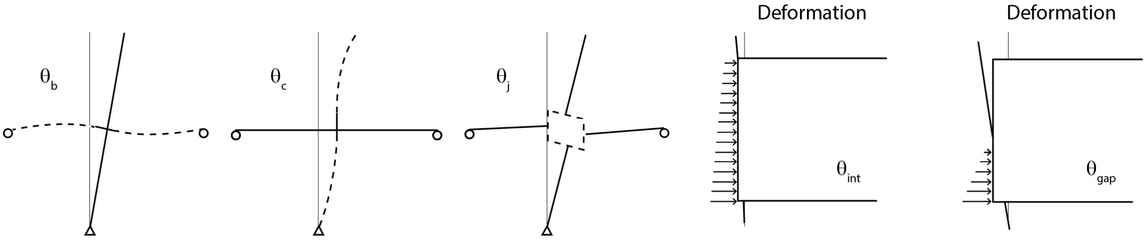


Figure 6. Rotation contributions to post-tensioned timber frames (modified from Smith et al,.2014).

The first three components shown in Figure 6 can be calculated through the use of common elastic deformation formulas while a detailed description of the evaluation of the interface compression deformation and the gap deformation can be found in (Smith *et al.*, 2014, Newcombe, 2007).

For each connection the elastic deformation can be evaluated with the following equation:

$$\theta_{y,i}=\theta_{b,i}+\theta_{c,i}+\theta_{j,i}+\theta_{int,i} \tag{1}$$

The total elastic deformation of the frame is weighted based on the proportion of moment capacity provided by each beam level:

$$\theta_y=\frac{\sum_{i=1}^{n} M_{con,i}\theta_{y,i}}{\sum_{i=1}^{n} M_{con,i}} \tag{2}$$

The total elastic deformation θ_y calculated has to be less than the inter-story drift limit equal to H/300 (i.e. 0.33%) for likely events, such as earthquakes and wind, with a return period equal to 25 years, according to the New Zealand Standard (NZS 1170.5, 2004) and Eurocode 8, 2004. The estimation of beam and column dimensions is then confirmed if the following equation is satisfied:

$$\theta_y<\theta_{SLS,lim} \tag{3}$$

Step 3. Post-tensioning design at SLS

The behavior of unbonded post-tensioned timber frames can be represented with a geometric non-linear force deformation plot. In the first branch of the bilinear curve two points of interest can be noted:

1. decompression point where compression in the extreme fibre is lost;
2. point where the centroid of the beam reaches decompression.

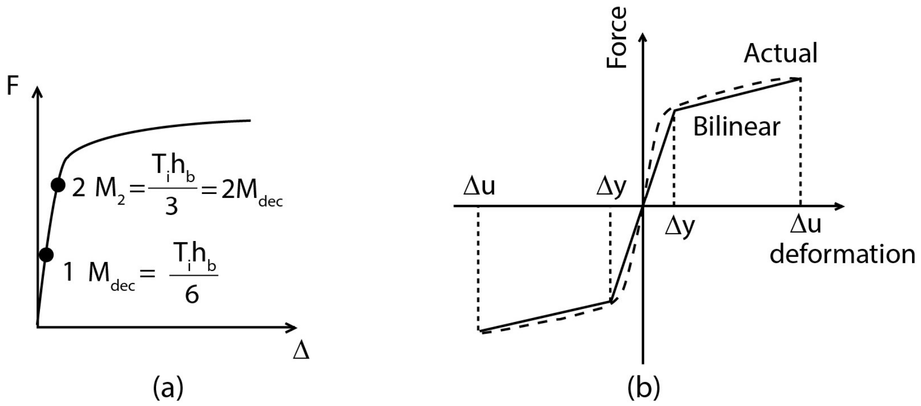


Figure 7. (a) lateral force-deflection curve and (b)bi-linear approximation of load-deflection curve (modified from Priestley and Tao, 2014).

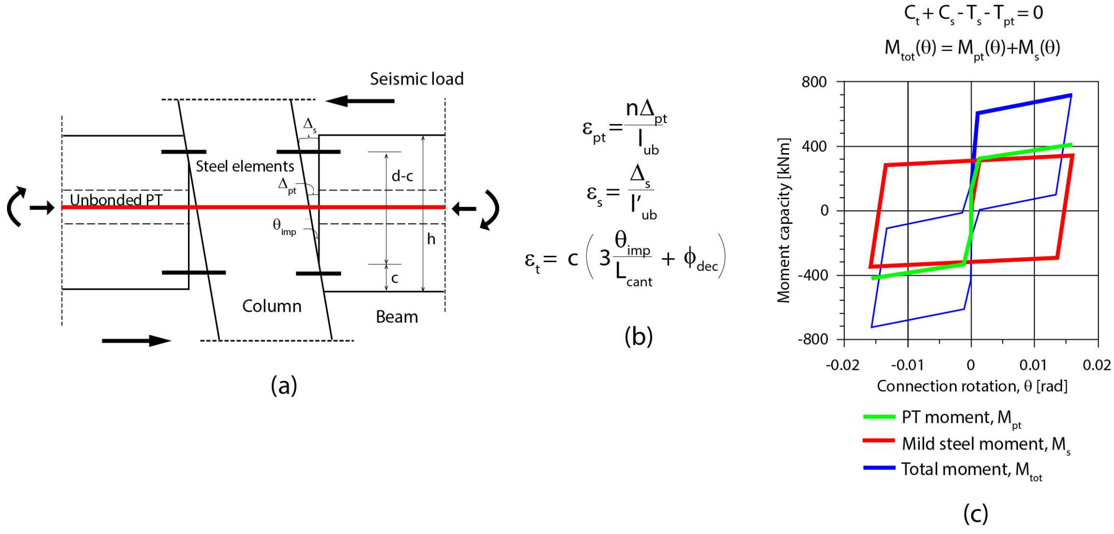
In order to guarantee that the yield point is not reached at SLS, post-tensioning is designed by imposing that 2M$_{dec}$ (point 2 in figure 7a) is equal to the moment demand at SLS obtained by DDBD:

$$2M_{dec}\cong M_{SLS} \rightarrow \frac{T_{pt,initial}h_b}{3}\cong M_{SLS} \rightarrow T_{pt,initial}=\frac{3M_{SLS}}{h_b} \tag{4}$$

Step 4. Check of the ULS moment at the imposed limit drift

Once section geometries, post-tensioning area A_{pt} and initial post-tensioning force $T_{pt,intial}$ have been selected, the capacity moment of the connection at ULS can be evaluated by the use of the Modified Monolithic Beam Analogy (MMBA) (Palermo, 2004). This consists of an iterative design procedure where a rotation at the connection is imposed, followed by an estimation of the neutral axis depth. The MMBA equation is applied to calculate the timber strain which can be used to calculate the timber compressive force. Using geometric equations the elongation of post-tensioning steel can be calculated. The section equilibrium needs to be checked in order to verify if the correct neutral axis depth was guessed. Once equilibrium is satisfied the moment capacity belonging to the imposed rotation can be calculated and the ULS check can be carried out by verifying that the moment capacity of the connection at the imposed ULS drift M_u is more than the connection moment demand obtained by DDBD procedure M_{ULS};

$$M_u \geq M_{ULS} \tag{5}$$



(a) (b) (c)

Figure 8. Section analysis for hybrid connection: (a) kinematics, (b) constitutive equations and (c) moment contribution and resultant flag-shape moment-rotation.

Step 4. Redundancy design: required mild steel reinforcement

Whatever the ULS check is satisfied or not mild steel reinforcement is added to ensure redundancy in the structure, especially for fire actions or unforeseen lost of post-tensioning.

Structural redundancy can be defined as the ability of the system to redistribute among its members the load which can no longer be sustained by some other damaged members (Biondini, et al., 2008). In post-tensioned timber frame, to ensure enough redundancy:

- moment capacity is provided by post-tensioned tendons and mild steel reinforcement;
- shear capacity is provided by corbels, dowel actions in the mild steel and friction due to the contribution of the post-tensioned tendons.

Primary effect of the addition of mild steel reinforcement is then the reduction of the probability of progressive collapse. For the dimensioning of mild steel in a hybrid connection a re-centring ratio β (defined as the ratio between post-tensioning moment and total moment capacity $\beta = \frac{M_{pt}}{M_{pt}+M_s}$) equal to 0.6 is suggested. When mild steel reinforcement is design for redundancy only (post-tensioning moment capacity is already greater then moment demand at ULS), a re-centring ratio β equal to 0.8 is arguably suggested as a good minimum requirement.

Case study building

Building description

To illustrate the frame design procedure, a case study building was designed (figure 8). The building is a further development of the design example proposed in (Pampanin, *et al*,.2013). The structure was designed to be located in Auckland on a type D soil (low seismic hazard, Z=0.13 for a 500 years return event). Columns and beams are made of Laminated Veneer Lumber (LVL) grade 16. Structural masses are reported in Table 1.

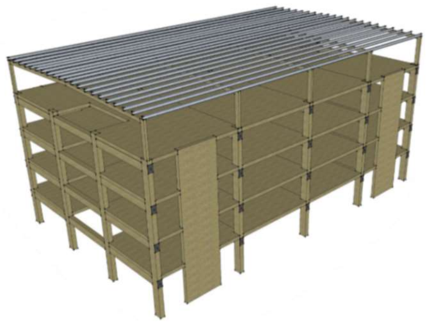


Figure 9. Render of the case-study building (modified from Granello et al., 2018).

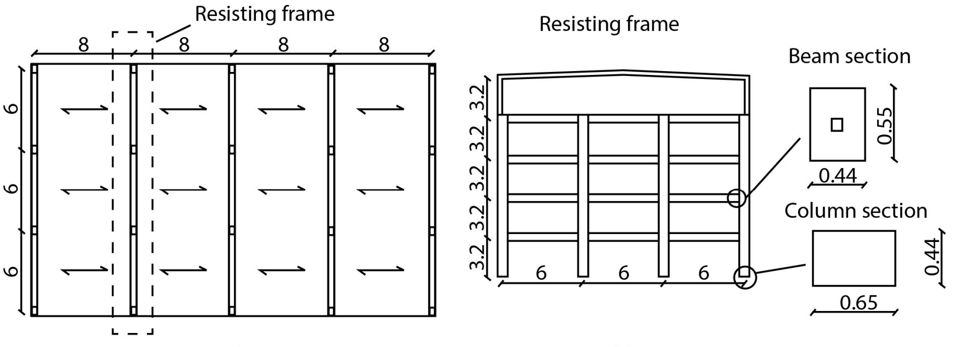
Floor	**$W_{i,total}$ [kN]**	**$W_{i,wall}$ [kN]**	**$W_{i,frame}$ [kN]**
4	2993	748	748
3	3044	761	761
2	3044	761	761
1	3044	761	761
tot	12125	3031	3031

Table 1. Seismic masses acting on the frame.

Application of the proposed design procedure

The dimensioning of section sizes and post-tensioning was carried out by using the proposed design procedure. Inter-storey drift limits were taken equal to 0.33% for seismic events with 25 years return period and equal to 2% for seismic events with 500 years return period. Members size and post-tensioning shown in figure 10 and Table 2 were selected to ensure that:

- the elastic connection rotation is less than the imposed limit drift at SLS,$\theta_y \leq \theta_{SLS,lim}$;
- the moment capacity of the connection at the imposed SLS drift is more than the connection moment demand obtained by DDBD procedure, $M_y \geq M_{SLS}$;



a) b)

Figure 10. a) plan view of the floor and b) lateral view of the building and member sections.

Storey	**Tendons number**	**Intial Post-tensioning [kN]**
1&2	2	330
3&4	2	205

Table 2. Detailing of post-tensioning.

The capacity moment of the connection at ULS was evaluated by the use of the monolithic beam analogy (MBA) (Pampanin *et al.*, 2001). The condition $M_u \geq M_{ULS}$ was found to be satisfied.

Modelling and performance levels

The moment-rotation behaviour can be implemented into lumped plasticity models (see Figure 11) 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).

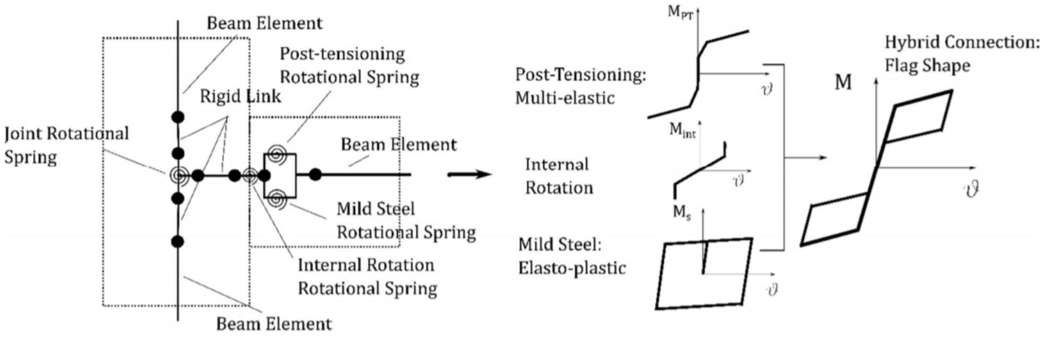


Figure 11. Modelling of the hybrid connection in timber elements (modified from Granello, 2018).

An ADRS analysis (Chopra and Goel, 1999) was carried out in order to evaluate the global performance of the prototype frame at Service Limit State (SLS) and Ultimate Limit State (ULS).

The pushover curve was evaluated using the analysis software OpenSees (McKenna, 2011). In the analysis, a triangular force distribution was used as a representation of seismic loading in accordance with New Zealand standards (NZS 1170.5, 2004). The connection behavior was simulated as described above with all other timber members being represented as elastic elements. The force–displacement pushover curve was then converted into an equivalent single degree of freedom acceleration displacement plot. The performance of the structure at both SLS and ULS was found to be satisfactory.

In figure 12 shows the numerical pushover curve obtained with the software OpenSees is compared with the lateral capacity curve of the frame analytically obtained.

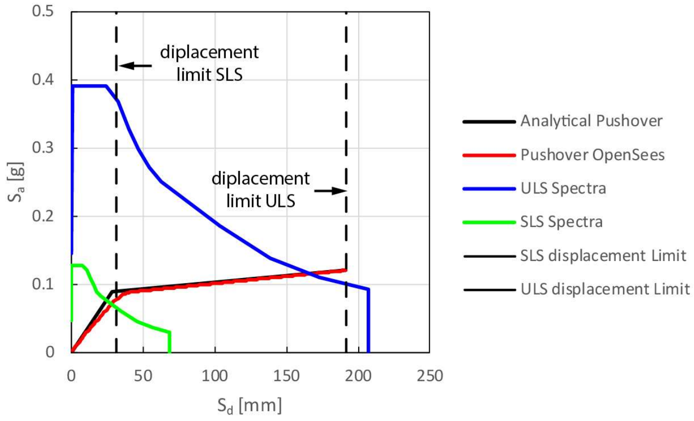


Figure 12. Comparison between analytical and numerical capacity curve..

Design for redundancy

Even if the performance of the structure was found to be satisfactory, mild steel reinforcement was added to ensure redundancy. As introduced, for the dimensioning of mild steel reinforcement in a hybrid connection a re-centring ratio β equal to 0.6 is suggested. For the case study building, being the mild steel reinforcement designed for redundancy only (post-tensioning moment capacity already greater then moment demand at ULS), a re-centring ratio β equal to 0.8 was arguably taken as good minimum requirement to reduce the probability of progressive collapse.

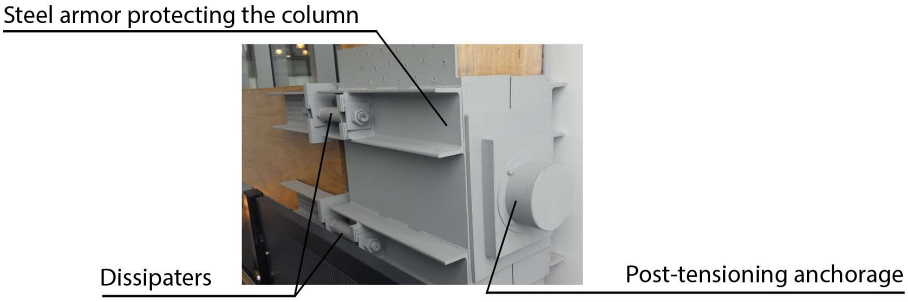


Figure 13. connection detailing.

Figure 14a shows the performance of the building designed with: β=1 , no mild steel reinforcement; β=0.8 , 4-10 mm diameter bars for level 1&2 and 4-8mm diameter bars for level 3&4 and β=0.6 , 4-17 mm diameter bars for level 1&2 and 4-15mm diameter bars for level 3&4.

The use of mild steel reinforcement allows to increase the capacity of the structure and reduce the seismic demand as effect of the hysteretic damping. Figure 13b shows how this allows to obtain a satisfactory performance of the building even in the case of an extraordinary event such as a fire (Horne *et al*., 2019), conservatively considered by assuming 70% losses in post-tensioning.

However, it's important to underline that this is an important effect of the use of mild steel reinforcement but it's not the primary effect that, as already discussed, is the increase in structural redundancy that reduces the probability of progressive collapse.

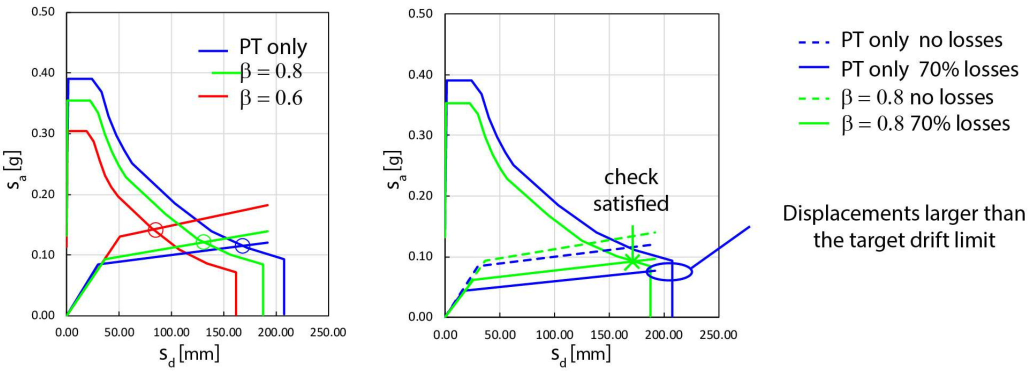


Figure 14. (a) Performance of the building designed with different values of re-centring ratio β and (b) effect of dissipation on the performance of the structure with 70% post-tensioning losses.

Conclusion

A design procedure has been proposed for the design of post-tensioned timber frames.

The procedure allows the determination of the required section sizes and post-tensioning forces to guarantee the performance objective at ultimate limit state (ULS) and at serviceability limit state (SLS) by achieving the lateral strength demand of the frame, without exceeding the total allowable displacement limits.

Dissipaters have been provided to ensure redundancy, if the post-tensioning losses occurs the expected ULS performance is guaranteed.

An application of the procedure for a case study structure located in Auckland (low seismic hazard) has been presented and the design method applied has been validated through the results of non-linear pushover analysis.

Further research is necessary to better quantify the effect of mild steel reinforcement on the performance of the structure and investigate the behaviour of the structures under dynamic non-linear analysis. However, the results obtained demonstrate that the proposed approach can be a valuable tool for design purposes.

References

Biondini, F., Frangopol, D.M. and Restelli, S. (2008) On Structural Robustness, Redundancy, and Static Indeterminacy. In: *Structures Congress 2008*]. 14 October 2008 Reston, VA: American Society of Civil Engineers. pp. 1–10.

Chopra, A.K. and Goel, R.K. (1999) Capacity-Demand-Diagram Methods Based on Inelastic Design Spectrum. *Earthquake Spectra*. 15 (4), pp. 637–656.

Comite Europeen de, N. (2004). *Eurocode 8 - Design of structures for earthquake resistance. Brussels, Belgium.*

Granello, G., Palermo, A., Pampanin,S.,Smith, T., Sarti, F., (2018). The implications of post-tensioning losses on the seismic response of pres-lam frames*. Bulletin of the New Zealand Society for Earthquake Engineering, Volume 51, Issue 2*.

Horne, P., Palermo, A., Abu, A., Moss P., (2019). Implications of seismic design detailing on the fire performance of Post-Tensioned Timber frames. *Pacific Conference on Earthquake Engineering.*

McKenna, F. (2011) “OpenSees: a framework for earthquake engineering simulation”. *Computing in Science & Engineering 13.4, pp. 58–66.*

Newcombe, M. (2007) *Seismic Design of Multistorey Post-Tensioned Timber Buildings*. Università degli Studi di Pavia, Pavia, Italy.Master Degree in Earthquake Engineering.

NZS 1170.5 (2004) *New Zealand Standard*.

Palermo, A. and Pampanin, S. (2004) The use of controlled rocking in the seismic design of bridges. *Doctate Thesis, Technical Institute of Milan, Milan*.

Palermo, A., Pampanin, S., Buchanan, A. and Newcombe, M. (2005) *Seismic Design of Multi-Storey Buildings using Laminated Veneer Lumber (LVL)*.

Pampanin, S., Palermo, A., and Buchanan, A. (2013) Post-Tensioned Timber buildings - Design Guide Australia and New Zealand. *Structural Timber Innovation Company.*

Pampanin, S. (2005) Emerging Solutions for High Seismic Performance of Precast/Prestressed Concrete Buildings. *Journal of Advanced Concrete Technology*. 3 (2), pp. 207–223.

Pampanin, S., Priestley, M.J.N. and Sritharan, S. (2001) Analitical modelling of the seismic behaviour of precast concrete frames designed with ductile connections. *Journal of Earthquake Engineering*. 5 (3), pp. 329–367.

Priestley, M.J.N. (1998) Displacement-based approaches to rational limit states design of new structures. *11th Eur. Conf. Earthquake Eng.: Invited Lectures (pp. 317-335).*

Priestley, M.J.N., Calvi, G.M. (Gian M. and Kowalsky, M.J. (2007) *Displacement-based seismic design of structures*. IUSS Press.

Priestley, M.J.N., Sritharan, S. (Sri), Conley, J.R. and Pampanin, S. (1999) Preliminary Results and Conclusions From the PRESSS Five-Story Precast Concrete Test Building. *PCI Journal*. 44 (6), pp. 42–67.

Priestley, M.J.N. and Tao, J.R. (2014) Seismic Response of Precast Prestressed Concrete Frames With Partially Debonded Tendons. *PCI Journal*. 38 (1), pp. 58–69.

Smith, T., Fragiacomo, M., Pampanin, S. and Buchanan, A.H. (2009) Construction time and cost for post-tensioned timber buildings.

Smith, T., Ponzo, F.C., Di Cesare, A., Pampanin, S., Carradine, D., Buchanan, A.H. and Nigro, D. (2014) Post-Tensioned Glulam Beam-Column Joints with Advanced Damping Systems: Testing and Numerical Analysis. *Journal of Earthquake Engineering*. 18 (1), pp. 147–167.

Sporn, B. and Pampanin, S. (2013) A “ retrofit ” solution for Force-Based Design: eliminating the need for iteration and initial period estimation. In: *New Zealand Society for Earthquake Engineering*. 2013 pp. 1–11.