SEISMIC DESIGN AND NUMERICAL VALIDATION OF POST-TENSIONED TIMBER FRAMES

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ABSTRACT:

A seismic design procedure and numerical sensitivity study of post-tensioned timber frames for multi-storey buildings is presented. The paper describes a displacement-based capacity design approach for jointed timber systems. Many aspects of the Direct Displacement-Based Design procedure for reinforced concrete are used for post-tensioned timber frames. An iterative design procedure is required due to the difficulty of predicting the elastic deformation and unique considerations are required for dynamic higher mode amplification of solid timber frames.

Numerical models are used to investigate the seismic response of post-tensioned timber frames, with lumped rotational springs and frame members subjected to a suite of far field and near field earthquakes. The results of the numerical analyses are compared with the design values. Simplified analytical design equations are given.

KEYWORDS: Timber, Post-Tensioned, DDBD, Seismic, Frame, Laminated Veneer Lumber (LVL)

INTRODUCTION

The implementation of Performance-Based Seismic Engineering (PBSE) into structural design practice has shown that different levels of structural damage and business downtime lead to financial losses. Depending on seismic intensity, informed decisions consider a life-cycle cost-analysis rather than only the initial construction costs [Krawinkler, 1999]. This has led to development of high performance structural systems capable of limiting damage to desired levels in a seismic event. Other life-cycle performance-based considerations are energy efficiency and sustainability, increasingly important in countries such as New Zealand that have ratified the Kyoto Protocol. Cost-benefit analysis of energy efficiency can lead to government subsidies (or 'carbon credits') from reduced carbon emissions [Buchanan, 2007; Perez et al., 2008].

These sustainability issues have resulted in renewed interest in timber construction in New Zealand, strongly supported by government. For typical light timber frame construction, only short spans are possible and the floor plan is often restricted because many walls are required for lateral load resistance [Thomas, 1991]. Solid timber construction allows larger spans and more open floor plans, as seen in buildings in New Zealand and Japan. Innovative new solutions have been proposed [Palermo *et al.*, 2005] suitable for multi-storey timber buildings in high seismic regions, either commercial or residential, with long spans, shallow beams, wide open spaces [Buchanan, 2008], high seismic performance, easy construction and economic viability [Smith, 2008].

The new timber building system is a product of emerging solutions developed for post-tensioned precast concrete construction (Figure 1a) in the U.S. PRESSS (Precast Seismic Structural Systems) Program at the University of California, San Diego [Priestley *et al.*, 1999]. The concept has also been extended to steel structures (Figure 1b) [Christopoulos *et al.*, 2002; Ricles *et al.*, 2001]. In all these systems, prefabricated structural elements are assembled on-site and connected with post-tensioned tendons and/or bars. Under seismic loading the inelastic demand is accommodated at connections with controlled rocking so that the structural elements remain essentially elastic and undamaged. Combining unbonded post-tensioning with energy dissipation (mild steel bars or other energy dissipators); a "hybrid" solution (Figure 1d) is obtained, with 'flag-shaped' hysteresis behaviour.

Jointed ductile timber systems (Figure 1c), termed PRESSS-Timber , Laminated Veneer Lumber (LVL) have been tested extensively at the University of Canterbury [Palermo *et al.*, 2005; Palermo *et al.*, 2006]. Brittle failures in previous studies [Fairweather and Buchanan, 1992] are eliminated by the low variability of LVL strength due to the staggered lamination of 3mm wood veneers. Recent analytical and numerical investigations [Newcombe *et al.*, 2008a] have concentrated on accurate modelling of post-tensioned timber connections, verified by numerous experimental subassembly tests. Within this contribution, the response of prestressed timber frames that incorporate the connection model, designed according to a modified displacement-based design approach [Priestley *et al.*, 2007], are investigated.

2. DIRECT DISPLACEMENT-BASED DESIGN OF PRESSS-TIMBER FRAMES

Direct Displacement-Based Design (DDBD) is a simplified seismic design procedure proposed developed over the last decade and most recently presented by Priestley *et al* [2007]. A displacement-based procedure, rather than a traditional forced-based approach, is most appropriate for jointed ductile connections, including PRESSS-Timber since the moment capacity of the connections is directly related connection rotation and thus to the interstorey displacement. Also, the code based displacement limits tend to govern the design and the high post-yield stiffness of the systems, due to the elastic elongation of post-tensioning, results in a high sensitivity of the base shear over the full range of displacement. Currently DDBD procedures are available for many forms of reinforced concrete [Priestley *et al.*, 2007], steel frames [Sullivan *et al.*, 2006], precast-concrete [Priestley, 2002] and light timber framing [Filiatrault *et al.*, 2002]. Modifications to the DDBD procedures are required for PRESSS-Timber. Details of the DDBD procedure are not introduced here; emphasis is given to key-design parameters affecting the PRESSS-Timber systems. The reader is referred to Priestley *et al* [2007] for information on the procedure.

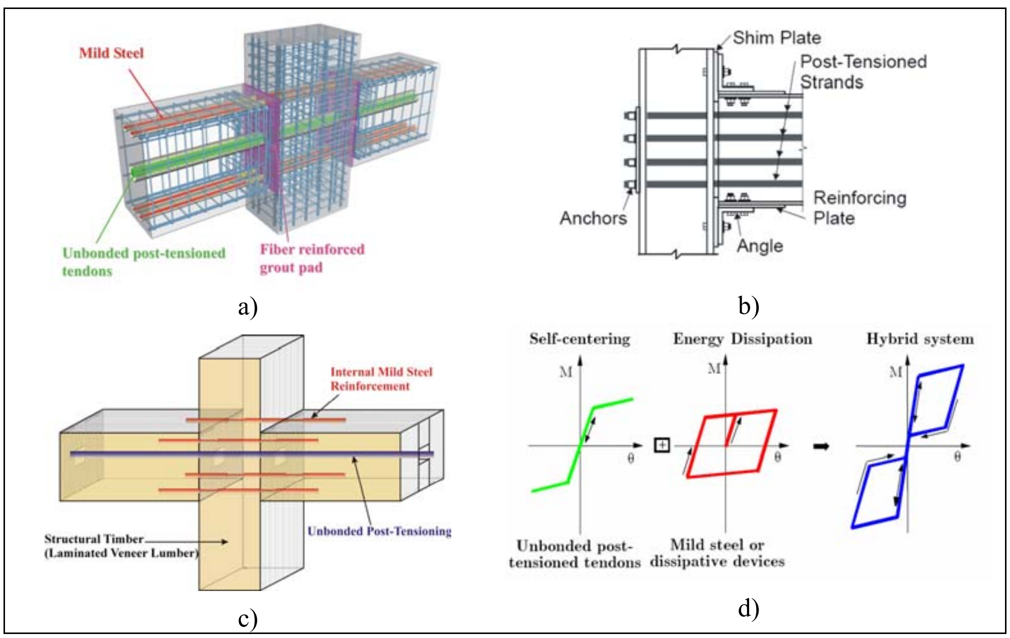


Figure 1 Jointed ductile rocking systems a) Precast concrete (c.o. S. Nakaki) b) Steel [Christopoulos *et al.*, 2002] c)Timber d) General Flag Shaped Moment-Rotation Response of a Hybrid Connection [fib, 2004]

2.1. Estimation of Yield Rotation

Prediction of the equivalent single degree of freedom yield rotation of a frame is a fundamental step in the DDBD design procedure. By accurately predicting the yield rotation, the structural ductility is known and iteration within the design procedure is not required. For reinforced concrete, the yield displacement of a system is essentially independent of its strength and stiffness [Priestley et al., 2007] but can be accurately related to geometry of the section and the material properties of the steel only. Unfortunately, for PRESSS-Timber frames the yield rotation is strongly dependent on strength. Therefore, estimates of system ductility in the design procedure may vary significantly and iterations may be required to ensure that the correct system ductility is achieved. Approximate formula can be derived in order to predict the yield rotation for typical frame geometries with reasonable accuracy which will in most cases reduce the number of iterations or negate the need to iterate in the design procedure.

2.1.1 Contributions to Yield Rotation

The significant contributions to the yield rotation for Timber-PRESSS frames are the elastic flexural and shear deformation of the beam and columns, the joint panel zone shear deformation and the elastic connection rotation, as expressed in Eqn. 2.1.

$$\theta_y = \theta_{b,y} + \theta_{c,y} + \theta_{j,y} + \theta_{imp,y} \tag{2.1}$$

where: $\theta_{b,y} + \theta_{j,y} + \theta_{c,y}$ is the elastic deformation of the beam, joint and column elements respectively and $\theta_{imp,y}$ is the deformation due to elastic rotation of the rocking connections.

The interstorey elastic deformation of a frame can be estimated by considering an interior beam-column subassembly (see Figure 2a) with points of inflexion at the mid height of the column and mid span of the beams. To illustrate the relative contributions of each deformation component, given in Eqn. 2.1, a case study 6-storey frame is considered (for further details see Newcombe *et al* [2008b]). From the DDBD of the case study frame the moment demands for the beam-column connections at the first level (390kNm) are calculated assuming that the connections are unarmored (with perpendicular to grain timber bearing on parallel to grain timber, as shown in Figure 2b) resulting in low axial stiffness and low system ductility. For the same frame geometry and reinforcement, an alternative connection arrangement is also considered where steel armoring reduces perpendicular to grain bearing stresses, thus increasing the axial stiffness of the connection (see Figure 2c). The deformation components of the armored and non-armored solutions are compared with an equivalent precast concrete beam-column subassembly (with an assumed cracked section stiffness of 0.5EI).

The most variable component for the 3 cases considered above (unarmored timber, armored timber and the concrete connection) is the elastic deformation of the connection. This is demonstrated in Figure 2d and Figure 2e where the relationship for the moment and neutral axis depth versus connection rotation is shown respectively. From the yield moment of each type of connection, the deformations of the beam, column, joint panel and the connection are computed and shown in Figure 2f. The most significant component for the unarmored connections is the connection elastic rotation followed by the joint panel deformation. For the armored connections the most significant component is the joint panel deformation. This is the effect of the low shear modulus of timber (approximately 600MPa) and the high shear forces generated by the post-tensioning within the joint region. In addition, the shear deformation of the timber members is non-negligible (>10%) and should not be ignored (as typically done for reinforced concrete).

It is not immediately evident from Figure 2d but the yield rotation of a connection can also be highly dependent on the applied axial load (or post-tensioning force). The effect of varying the axial load on the yield rotation of the connection is shown in Figure 2g for the timber case study subassemblies: with and without armoring. For unarmored timber the yield rotation of the connection is extremely variable with axial force while armored timber is fairly insensitive to axial load.

2.1.2 Derivation of expressions to approximate frame yield rotation

For a general case, the system deformation of a frame can be extrapolated from the deformation of an internal beam-column subassembly. It is assumed that the same member sizes are used for most of the building height, the beam section capacity can be related to the critical connection capacity, LVL is used and that there is a fixed ratio between the moment contribution for the post-tensioning and energy dissipation. The variation of the yield moment up the height of the case study frame is given in Figure 2h and the variation of the connection yield rotation up the building height is given in Figure 2i.

Considering the elastic member deformation up the frame and connection yield rotation predicted by an analytical moment-rotation analysis (calibrated to experimental results), with further geometrical simplifications,

approximate empirical equations (see Eqn. 2.2 and Table 2.1) are generated for the system yield rotation of a frame for armored and unarmored connections. These expressions are in terms of the bay length, $\mathbf{L_b}$, and the beam height, $\mathbf{h_b}$ and aim to reduce the number of iterations required for the DDBD.
For the two types of timber connections considered, the size of members is not dictated by the ultimate strength requirements but the elastic deformation limits. Hence, even if an elastic design is performed, the elastic deformation of the frame may exceed the design drift limit (1.5-2.5%). However, it is noted that these drift limits may not necessarily be applicable to elastic responding frames. Therefore, Eqn. 2.2 under an elastic design will also indicate whether the design drift limit is likely to be exceeded.

$$\theta_y = \left(\theta_{b,y} + \theta_{c,y}\right) + \theta_{j,y} + \theta_{conn,y} = C_1\left(\frac{L_b}{h_b} + 60\frac{h_b}{L_b}\right) + C_2\left(C_3 - \frac{h_b}{L_b}\right)\left(1 - \frac{3h_b}{L_b}\right) + C_4\left(1 - \frac{h_b}{L_b}\right) \quad (2.2)$$

Table 2.1 Coefficients for empirical yield rotation formulation

Frame Type	C_1	C_2	C_3	C_4
No armoring	0.00041	0.0075	1.025	0.0077
Steel armoring	0.00094	0.0131	0.980	0.00175

2.2. Evaluation of the Equivalent Viscous Damping Relationships

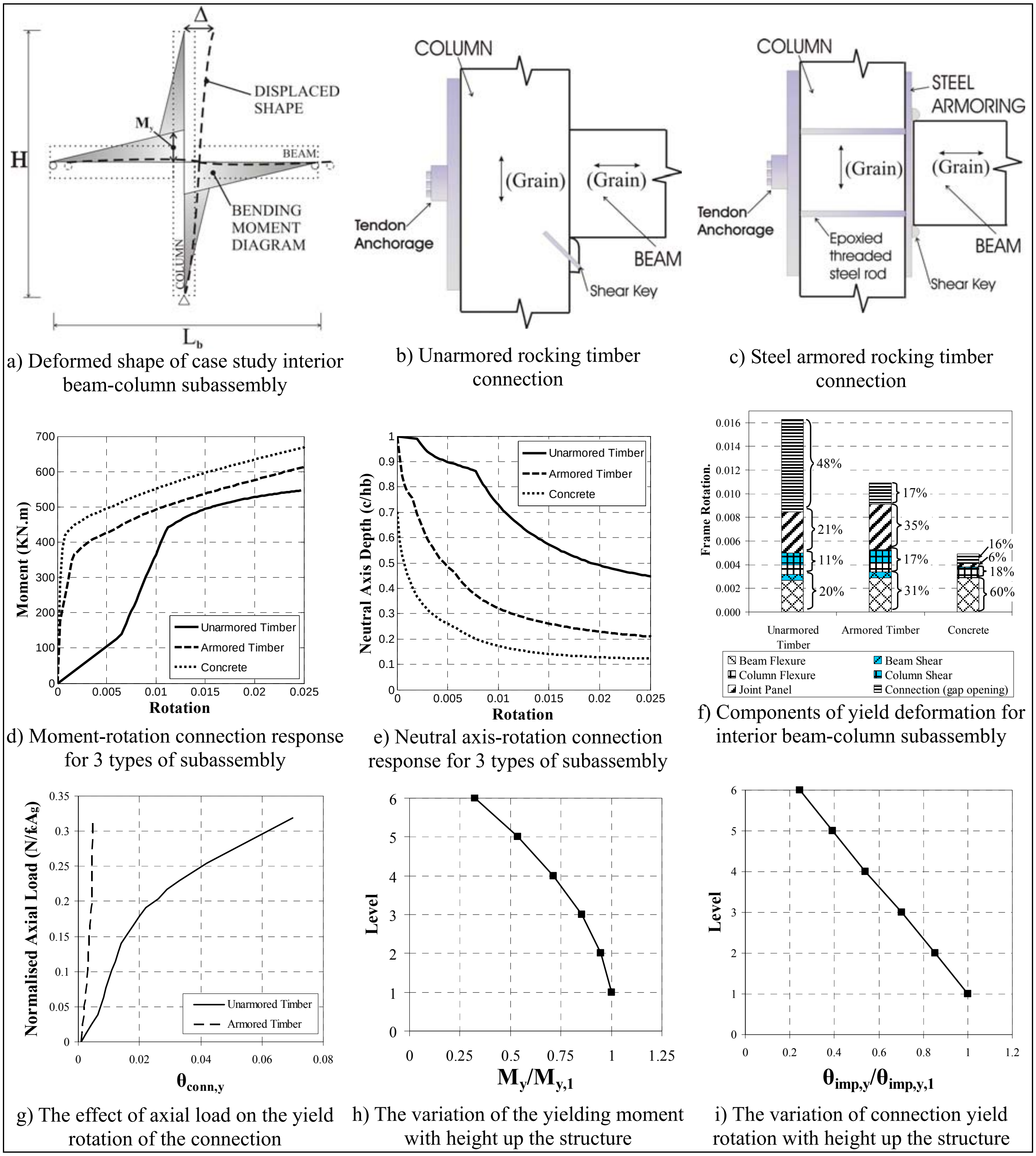
Under the displacement-based design procedure, equivalent viscous damping is typically expressed as a function of the ductility (**μ**). As in Priestley et al., [2007], the equivalent viscous damping (EVD) is the sum of the elastic and hysteretic damping. Given the novelty of the system the elastic or intrinsic damping associated with post-tensioned timber frames is under investigation and requires dynamic experimentation. Previous research on light timber frame buildings [Durham *et al.*, 1999; Filiatrault *et al.*, 2002; Foliente, 1995] indicates that 2% of critical damping is appropriate, if conservative. The hysteretic component of damping in DDBD is derived from the energy absorbed by inelastic response. This can be approximated by using relationships from Priestley et al., [2007] and the area captured within the frame system hysteresis at maximum response. Note that the shape of the system hysteresis depends on the axial stiffness of the connections. For further information on damping for PRESSS-Timber, refer to Newcombe *et al.,*[2008b].

3. NUMERICAL INVESTIGATION AND VALIDATION OF FRAME DESIGN PROCEDURE

3.1. Design parameters and modeling approach

The accuracy of the proposed design procedure for PRESSS-Timber frames, with unarmored connections, is verified using inelastic time-history analysis (ITHA) on four two-dimensional frame geometries, based on a case study structure (see Figure 3a), using the finite element modeling program RUAUMOKO [Carr, 2006]. The frames are either 6 or 10 storeys, by 5 or 8 bays, with the same plan dimensions (total length 36m) and interstorey height (3.8m). In absence of alternative methodology, it was determined if the amplification and modification factors for the higher mode response of typical reinforced concrete frames [Priestley *et al.*, 2007] are applicable for timber frames. The design procedure was verified for the design level earthquake and the maximum credible earthquake (MCE). Fifteen earthquake records (10 far field and 5 near field), scaled to the 2% damped design acceleration spectrum for Wellington [NZS1170.5:2004], were applied to each of the frames. All frames were designed with 2% design drift limitation so that P-Δ effects could be ignored.
A simplified modeling approach was used with lumped rotational springs to simulate the connection response; a multi-linear elastic model for the moment contribution from the post-tensioning and a bi-linear inelastic model for the energy dissipation as shown in Figure 3a. This simplified model does not take into account geometric beam elongation which will contribute to column demands. However, the soft connections often result in neutral axis depths larger than the half height of the section; theoretically this will result in negligible beam elongation due to the gap opening. To reduce the complexity of the model the joint panel zone deformation was ignored

(and was subtracted from Eqn. 2.2 for the DDBD). Throughout all the frames, the ratio of the connection moment generated by the post-tensioning (and axial load) versus the moment generated by the energy dissipation (λ ratio) of 1.4 was maintained to ensure the frame has re-centering ability. For further information the reader is referred to Newcombe *et al.*,[2008b].



a) Deformed shape of case study interior beam-column subassembly

b) Unarmored rocking timber connection

c) Steel armored rocking timber connection

d) Moment-rotation connection response for 3 types of subassembly

e) Neutral axis-rotation connection response for 3 types of subassembly

f) Components of yield deformation for interior beam-column subassembly

g) The effect of axial load on the yield rotation of the connection

h) The variation of the yielding moment with height up the structure

i) The variation of connection yield rotation with height up the structure

Figure 2 Elastic deformation of PRESSS-Timber frames

3.2. Summary of results

Once the frames were designed according to the modified DDBD procedure, an adaptive push over analysis was performed to verify the system yield rotation of the frames. Often the value assumed for DDBD, from Eqn. 2.2, was satisfactory but slightly conservative. For the frame designs, it was not reasonable to maintain a constant λ ratio of 1.4 for the beam-to-column connections. At the lower floors a smaller λ ratio was required due to higher axial forces (due to post-tensioning) and moment demands, resulting in a high neutral axis and lower moment generated from the post-tensioning. For higher levels the λ ratio could be increased. These variations in λ ratio affect the hysteretic energy absorption at each level, as illustrated by Figure 3b, giving the hysteretic loops for the connections at the 1st and 9th level of the 10 storey-5 bay frame designed to a MCE.

The current inelastic deformed shape used for the DDBD of reinforced concrete frames [Priestley *et al.*, 2007] appears to be appropriate for PRESSS-Timber. The mean and mean $\pm 1\sigma$ (standard deviation) values compared with the design displacement profile are plotted Figure 3c for the 10 storey-5 bay frame for the design level intensity.

The interstorey drift profiles obtained from the ITHA are, in most cases, within $\pm 1\sigma$ of the design limit up the entire height of the structure (resulting in an efficient use of the allowable rotation at every floor). This is illustrated for the 10 storey-5 bay frame at design level intensity in Figure 3d.

Also, the capacity design shear envelope, recommended by Priestley *et al.*, [2007], for reinforced concrete and shown here in Eqn. 3.1 appears satisfactory for PRESSS-Timber DDBD. For all frames, at almost every level, the mean $\pm 1\sigma$ response from the ITHA is within the capacity design shear envelope (see Figure 3e).

However, the capacity design interstorey moment envelope used for the DDBD of reinforced concrete, recommended by Priestley *et al.*, [2007], seems inappropriate for PRESSS-Timber. For reinforced concrete it is reasonable that some of the interstorey moment capacities can be exceeded because the columns have sufficient ductility capacity and moment redistribution can occur. However, for timber the columns have no flexural ductility. Therefore, more conservative capacity design expressions are proposed for the DDBD of PRESSS-Timber (see Eqn. 3.1 and Figure 3f). In addition, it can be observed in Figure 3f that the capacity design moment envelope significantly exceeds the ITHA values at lower levels. This is a result of larger amounts of hysteretic damping at those levels (see Figure 3b). Hence, the reduction in moment demand at lower levels is simply an effect of the variation in the λ ratio.

$$\phi_f M_N \geq \phi^0 \omega_f M_E \tag{3.1}$$

where: $\phi_f M_N$ is the reduced nominal moment capacity of the columns, ω_f is the dynamic amplification factor for flexure due to higher modes and μ^o is the ductility at overstrength response (taken as **μ** for this investigation). and:

$$\omega_f = \begin{cases} 1.0 & \text{At the roof and base level} \\ 1.66 + 0.13\left(\mu^0 - 1\right) & \text{All other levels} \end{cases} \tag{3.2}$$

A comparison of the DDBD design parameters and the ITHA results is given Table 3.1 for the 10 storey-5 bay frame for two intensities. The ratio of the mean ITHA response and DDBD values show that interstorey moment is the most sensitive to higher mode amplification, followed by interstorey drift and then shear. The results also indicate that there is larger variation in the results for higher intensities.

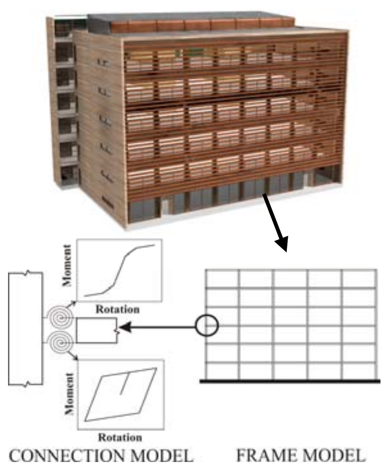
At this point a simplification in the capacity design considerations is possible. Unlike reinforced concrete the shear and flexural capacity of a solid timber member are directly related. By assuming common properties of laminated veneer lumber, Eqn. 3.3 can be used as guide to indicate whether the capacity design moment ($\phi^o \omega_f M_E$) or capacity design shear demands will govern the section size of the column. If the capacity design moment governs the ratio of the interstorey height (ΔH) to the in-plane depth of the column (h_c) exceeds 3.0:

$$\frac{\Delta H_n}{h_c} \geq 3.0 \tag{3.3}$$

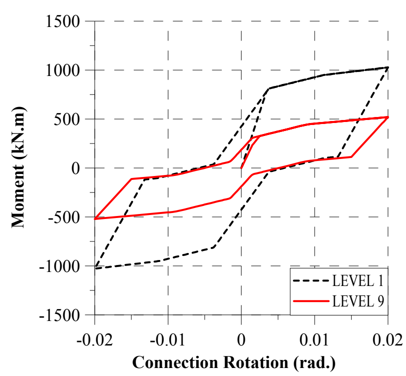
3. CONCLUSIONS

The Direct Displacement-Based Design philosophy developed for reinforced concrete has been modified, validated and successfully applied to post-tensioned timber buildings. Major considerations include the following:

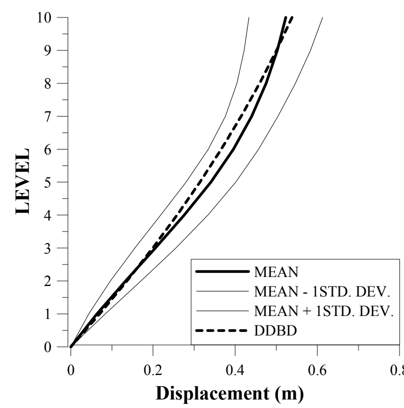
- There are several deformation components, usually neglected in concrete design, which cannot be neglected for post-tensioned timber frames, the joint panel zone deformation being the most significant.
- The elastic connection deformation is highly dependent on the connection's axial stiffness (hence whether or not armoring is used to protect the face of the column from high stresses perpendicular to the grain). The elastic deformation of soft connections is also highly dependent on the level of axial load in the beam.
- New approximate empirical equations developed for DDBD of post-tensioned timber frames can be used to reduce the number of iterations in the design procedure and provide a conservative check that drift limitations are not exceeded for an elastic design.
- An initial method for determining the equivalent viscous damping of post-tensioned timber frame buildings is proposed. However, it is recognized that further experimental research is required to more accurately characterize the elastic damping of solid timber frames and wood in general.
- A numerical investigation of post-tensioned timber frame buildings, designed according to the modified DDBD procedure, has shown that most of the current amplification factors and modification factors used to account for higher modes in reinforced concrete frames can be used for timber. For the higher mode amplification of interstorey moment it is recognised that greater conservatism is required due to the brittle nature of timber in flexure.



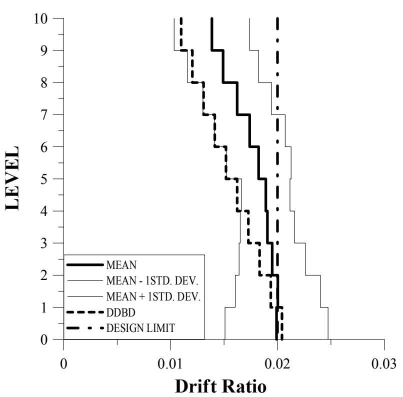
a) Case study structure; UoC Biological Sciences Building (c.o. N. Perez)



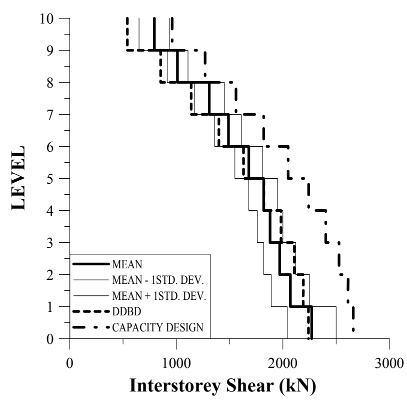
b) Connection hysteresis at level 1 and 9 for the 10-storey frame (at MCE)



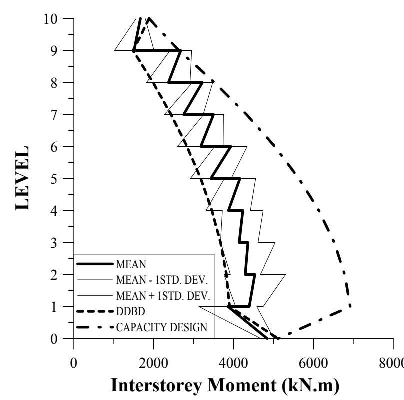
c) Displacement profile for 10 storey frame



d) Interstorey drift profile for 10 storey frame



e) Interstorey shear profile for 10 storey frame



f) Interstorey moment profile for 10 storey frame

Figure 3 Design procedure verification on case study structure

Table 3.1 Variation of the DDBD parameters, interstorey drift (θ), interstorey shear (V) and interstorey moment (M) from the time history analysis results for the 10 storey – 5 bay frame

	$\theta_{avg}/\theta_{DDBD}$		V_{avg}/V_{DDBD}		M_{avg}/M_{DDBD}	
	Mean	**Std. Dev.**	**Mean**	**Std. Dev.**	**Mean**	**Std. Dev.**
Design Level	1.151	0.098	1.075	0.158	1.200	0.221
MCE	1.414	0.195	1.174	0.237	1.384	0.274

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