Simplified Design Procedures for Post-Tensioned Seismic Resistant Timber Walls

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SUMMARY:
Focusing on a range of standardized sections, this paper provides an overview of simplified design tools developed for post-tensioned Laminated Veneer Lumber (LVL) walls either for the overall seismic design of the structural system, based on Displacement-Based considerations, and for design detailing of the rocking connections, through a section/connection analysis approach. In particular, referring to a moment-rotation behaviour of the critical rocking section/connection, a performance based approach is herein proposed by checking critical performance levels.
The intent is to provide practitioner with simplified whilst reliable design tools for a first preliminary design of the wall reinforcements and a quick check of the serviceability and ultimate limit states criteria. Two different charts are used to evaluate the post-tensioning and mild steel bars/devices respectively. The final part of the paper presents a design example for a case-study building, based on the application of the above mentioned design tools.

Keywords: seismic design, post-tensioning, timber walls, design charts

1. INTRODUCTION

Timber construction has been and is being widely used for residential buildings such as apartments, hotels etc., where short spans are required and several walls can be positioned within the building without interfering with the architectural layout. Among different construction systems, panel construction and solid panel construction systems are the most widespread.

In panel construction, the most common form of lateral force resisting system is the use of nailed walls (see Fig. 1.1a-b). These walls have light timber framing, with sheathing of plywood (Buchanan, 2007). Research carried out by (Stewart, 1987, Deam, 1996) showed plywood sheathed walls provide good hysteretic damping (10-15%), but the extensive pinching behaviour due to nail slip severely reduced the overall stiffness of the system. Similar systems have been tested in the United States (Pei *et al.*, 2012), and the results have shown similar damage to the wall nailed connections.

An alternative construction technology for multi-storey timber buildings is solid timber construction (Fig. 1.1c). Cross-laminated timber panels carry the loads down to the foundations. Large-format elements act as plates and carry horizontal loads; additional wind girders are therefore unnecessary. Thanks to the cross-banded lay-up, such elements can carry loads in both directions when used horizontally, and elements placed vertically can be used as free-standing components (Kolb, 2008).
Quasi-static and dynamic test results highlighted that the layout and design of the joints is strongly influencing the overall behaviour of the structural system. All forces and displacement are concentrated on a rather small region of the panel which can leading to local failure phenomena; therefore, it is confirmed that all the dissipated energy is resulting from the connections and the ductility capacity of the system is limited or controlled by the detailing of these regions (Ceccotti *et al.*, 2006, Ceccotti, 2008).

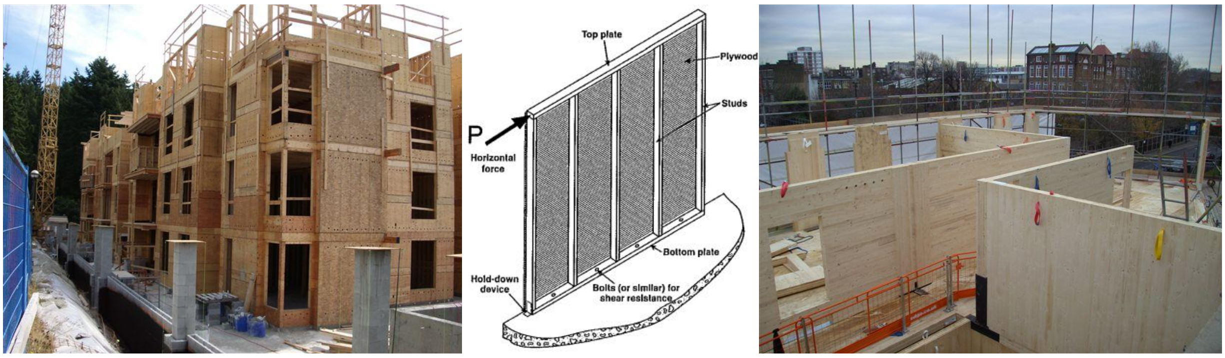


Figure 1.1. (a) Light timber framing building in Canada (c/o A. Buchanan). (b) Typical shear wall geometry (Buchanan, 2007). (c) Cross-Laminated building in London (c/o Andy Buchanan)

Alternative technologies capable of achieving high-performance minimizing structural damage were developed and introduced in the 1990s as main outcome of the U.S. PRESSS (PREcast Seismic Structural System) program coordinated by the University of California, San Diego and culminated with the pseudo-dynamic test of a large scale five-story test building (Priestley *et al.*, 1999). Among different connection solutions developed in the PRESSS program, the hybrid system (Fig. 1.2a) proved to be a promising and efficient solution. The system relies upon the combination of self-centering and dissipation contributions, provided by unbonded post-tensioning and mild steel reinforcement respectively. The advantages of this connection are: no damage in structural member, no post-residual displacement of the structure after the event, replaceability of the dissipaters, which is the only sacrificial part of the system.

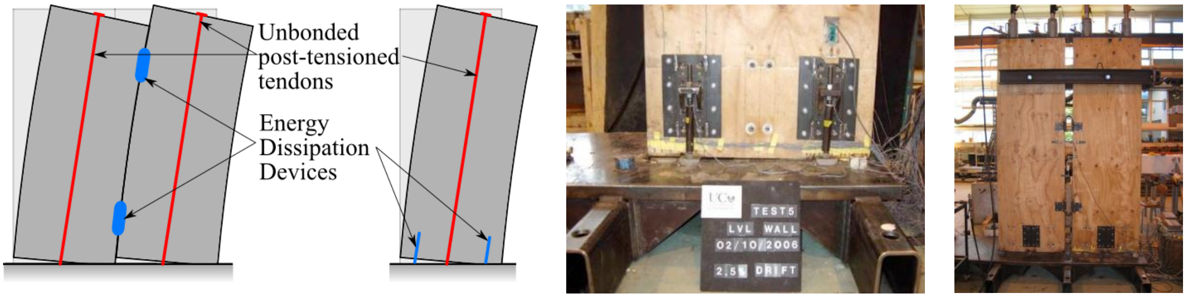


Figure 1.2. (a) hybrid concept for wall systems, modified from (NZCS, 2010); (b) hybrid wall with external dissipaters (Smith *et al.*, 2007); (c) coupled walls with UFP devices (Iqbal *et al.*, 2007)

The concept of hybrid system is material independent and it has subsequently been extended to steel structures (Christopoulos *et al.*, 2002), and to timber (engineered wood) structural systems (Palermo *et al.*, 2006a). An extensive overview of the technology applied to any structural material is summarized in (Canterbury Earthquake Royal Commission, 2011). This extension brought to new structural systems, referred to as Pres-Lam (Pre-stressed Laminated) system, which consist of large timber structural frames or walls made of any engineered wood products, such as LVL, glulam, Cross-Lam (CLT) etc.

Extensive experimental studies carried out at the University of Canterbury investigated different reduced scale post-tensioned timber wall solutions: single walls with either internal or external hysteresis or viscous dissipating devices, and coupled walls with U-shaped Flexural Plates (UFP) (Palermo *et al.*, 2006b, Iqbal *et al.*, 2007, Smith *et al.*, 2007). Each wall solution confirmed an excellent seismic performance with negligible level of structural damage limited to the yielding of dissipaters, which, if external, can be easily replaced.

Following the extensive research and development campaign, few buildings have already been constructed adopting the Pres-Lam technology.

The world's first commercial building which adopted this technology is given by the NMIT (Nelson Marlborough Institute of Technology) building (Fig. 1.3a), constructed in Nelson (Devereux *et al.*, 2011). The lateral loading system consists of post-tensioned coupled walls in both main directions. Steel U-shaped Flexural Plates (UFP) (Kelly *et al.*, 1972) link the pairs of structural walls together and provide additional overturning moment contribution and dissipative capacity to the system.
The Carterton Events Centre (Fig. 1.3b), located 100km north of Wellington, is the second building in the world adopting the Pres-Lam system (Palermo *et al.*, 2012). Single post-tensioned rocking walls were designed as the lateral load resisting system. Two post-tensioned high-strength bars (40mm diameter) provide re-centering contribution, and supplemental dissipation is given by internally epoxied mild steel rods.
Few other buildings using either Post-tensioned timber walls, and/or frames are either under construction or in the final design stage (Canterbury Earthquake Royal Commission, 2011).

(a) (b)

Figure 1.3. . (a) NMIT Building (b) Carterton Events Centre

Whilst the advantages of the systems in terms of seismic and structural performance are being recognized, the short-term cost-effectiveness of Pres-Lam systems has potential for significant improvement, which would ultimately lead to a wider adoption and implementation in the construction practice. Previous studies (Smith *et al.,* 2009) highlighted that the increased cost of prefabricated timber elements, in comparison to steel or concrete counterparts, comes mainly from the cost of the material, yet a significant portion of the total cost is represented by manufacturing costs which lacks of a properly implemented prefabrication process similarly to prestressed concrete industry.
On the other hand, buildings implementing these systems can bring some major benefits. Among those the lighter weight of the material brings important advantages in the design (reduced seismic mass, foundations) and in construction (crainage, speed, etc.). Needless to remember that higher speed of construction result into significant reduction in cost.

The present paper suggests suitable standard sections for post-tensioned timber walls. This standardization process is required for a full "prefabrication" process which would cut costs.
On the basis of a range of standardized sections proposed simplified design tools for post-tensioned single LVL walls, developed through extensive parametric moment-rotation analyses, are provided.
The simplified procedure is then implemented into a displacement-base design approach in the final part of the paper.
In the conclusive part, a worked design example on a case study building is provided to show the application of proposed method, incorporated in a displacement-based design procedure (Priestley *et al.*, 2007).

2. DESIGN CHARTS METHOD

The present section shows a preliminary design method developed an extended parametric analysis based on a moment-rotation procedure (see §2.1).

A rocking section with unbonded reinforcement, such as post-tensioning and debonded mild steel, requires violates conditions of strain compatibility, which is typically assumed in a section analysis approach, between steel and concrete (or timber). The strain incompatibility of the materials requires the introduction of an additional member compatibility equation to the two equilibrium conditions. The main steps of the procedure are briefly summarized below.

2.1. Moment-rotation analysis procedure

The moment-rotation step-by-step procedure for hybrid connections (Fig. 2.1a), originally proposed for precast concrete by (Pampanin *et al.*, 2001), modified by (Palermo *et al.*, 2008) and later expanded to timber by (Newcombe *et al.*, 2010) is herein adopted. A brief summary is given in this section. The procedure, shown in (Fig. 2.1a), is very similar in terms of general flow-chart to a standard moment-curvature sectional analysis used for concrete sections.

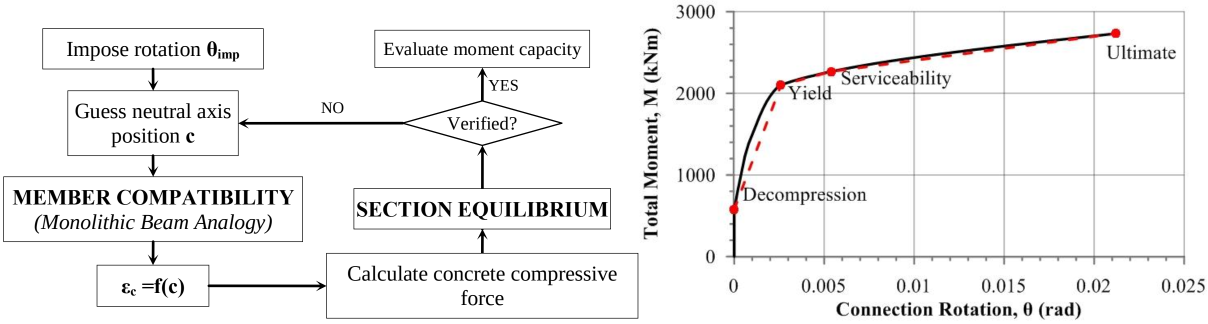
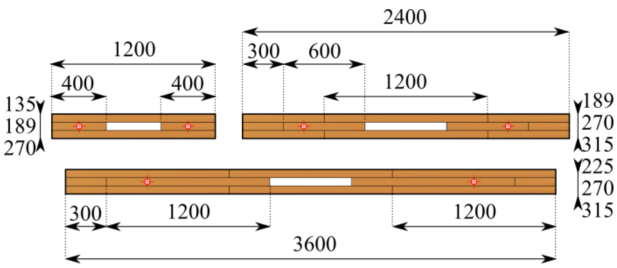


Figure 2.1. (a) schematic flow chart of the procedure (modified from (Pampanin, 2000)). (b) moment-rotation chart linearization

When deriving the moment-rotation curve, similarly to what done for concrete sections (Palermo, 2004) and according to the definitions given by (Priestley *et al.*, 1998) and (Kowalsky, 1997) performance deformation limits are defined (Fig. 2.1b): the *Decompression* point, corresponding to the gap opening; the *Yielding* point corresponding to the yield of the mild steel bars (ε_s=ε_{sy}); the *Serviceability* point corresponding to the achievement of 1% strain of the mild steel bars or timber strain ε_t = 0.002; the *Ultimate* point corresponding to the achievement of 6% strain in the mild steel bars or timber strain ε_t = 0.004.

2.2. Walls standard sections

Since LVL is provided in standard profiles (1200mm panel width which is then cut in different sizes), a range of standardized wall sections is herein proposed to be adopted; consequently, the depth of the wall is 1200mm, 2400mm or 3600mm (Fig. 2.2).



Width×Depth (mm)		
135×1200	189×2400	225×3600
189×1200	270×2400	270×3600
270×1200	315×2400	315×3600

Figure 2.2. Standardized wall sections

Each wall section is made of standard sections provided by manufacturers. The thickness of the proposed standardized walls is 400mm, 600mm and 1200mm, based on the use of 45mm, 63mm, 90mm and 105mm panels to build up the sections. Each wall is thus comprising either three or five panels, with different thickness depending on the void space needed for the insertion of the post-tensioning bars (see Fig. 2.2).

2.3. Materials

Material characteristics of timber as well as of post-tensioining and mild steel are reported in Table 2.1. Different grades of Laminated Veneer Lumber are available on the market. The material properties of LVL most commonly used and considered in the present work are reported below.

Table 2.1. (a) LVL material properties. (b) Post-tensioning and mild steel material properties

Property		Unit	Char. Value
Modulus of Elasticity	E	GPa	11
Bending	f_b	MPa	48
Tension parallel to grain	f_t	MPa	30
Compression perp. to grain	f_p	MPa	12
Compression parallel to grain	f_c	MPa	45
Shear in beam	f_s	MPa	6

(a)

Property	Unit	Char. value		
		Strands	MacAlloy	Mild Steel (Grade300)
Mod. of Elasticity	GPa	200	170	210
Yield stress	MPa	1520	835	300
Ultimate stress	MPa	1860	1030	375

(b)

Either threaded bars or strands can be used as post-tensioning reinforcement. As non-prestressed reinforcement or additional dissipation devices, either internal or external dissipaters (Fig. 2.3) could be used.

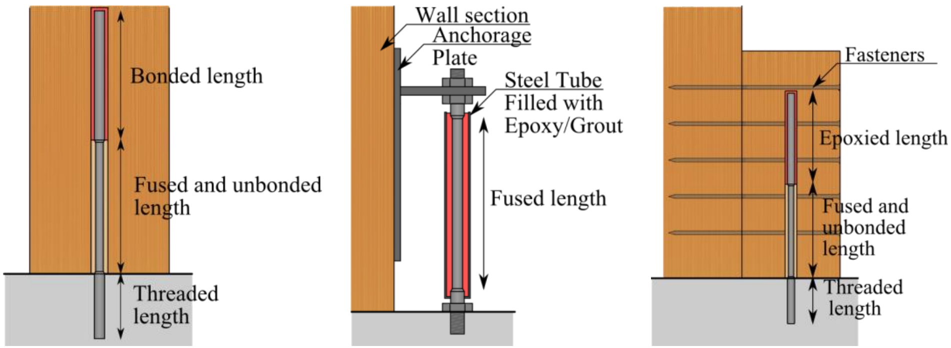


Figure 2.3. Mild steel dissipation devices: (a) internal bar; (b) replaceable fused bar; (c) replaceable internal bar

2.4. Structural and sectional parameters

For a given section, as shown in Fig. 2.4, dimensionless parameters are considered in the following analysis to obtain a general tool capable of evaluating moment capacity for a wide range of solutions.

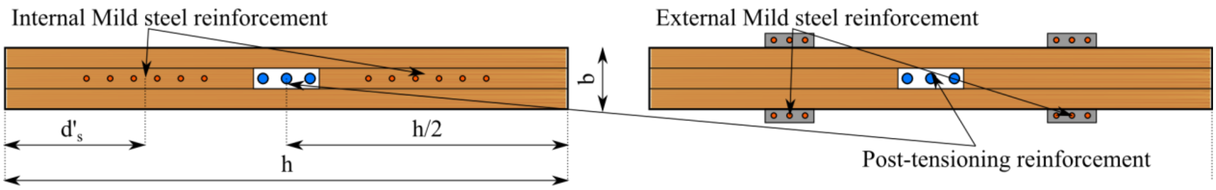


Figure 2.4. Wall general section with (a) internal and (b) external mild steel bars

Two different moment contributions can be distinguished: post-tensioning and axial, and mild steel contributions. Each of them is defined through the re-centering ratio λ, defined in Eqn 2.1.

$$\lambda = \frac{M_p + M_N}{M_s} \rightarrow M_s = \frac{M}{1+\lambda}; M_p + M_N = \frac{\lambda M}{1+\lambda} \tag{2.1}$$

Where M_p, M_N, M_s are y the post-tensioning, the axial load and the mild steel moment contributions, respectively

Dimensionless axial force and moments contributions, as well as mechanical reinforcement ratios, reported in Eqn. 2.2 are used as parameters.

$$\nu = \frac{N}{bhf_t}; \mu_s = \frac{M_s}{bh^2 f_t}; \mu_p + \mu_N = \frac{M_p + M_N}{bh^2 f_t}; \omega_s = \frac{A_s f_s}{bhf_t}; \omega_p = \frac{A_p f_p}{bhf_t} \tag{2.2}$$

Where b is the width and h the depth of the wall; f_t, f_s and f_p are the characteristic strengths of timber (in compression), mild steel and post-tensioning steel (in tension) respectively; ω_s, ω_p, A_p and A_s are the post-tensioning and mild steel reinforcements ratios and areas.

The design tools consist of a set of two charts (Fig. 2.5). The first chart in Fig. 2.4a (left) plots the post-tensioning reinforcement design moment against axial force for different post-tensioning steel mechanical ratios. This is similar in concept to a typical M-N interaction diagram used for the design of reinforced concrete columns. The initial post-tensioning stress is varied as third parameter: the top bold line represents the moment capacity of the section at a post-tensioning initial stress of $0.85f_{py}$; instead, thinner lines in the same shaded area represent decreases in tendon stress in steps of 5% of the bar yield stress for the same reinforcement ratio. The second chart on Fig.2.4b reports the mild steel moment contribution against the mild steel reinforcement ratio; edge distance (d_s) of the reinforcement bars is varied as third parameter.

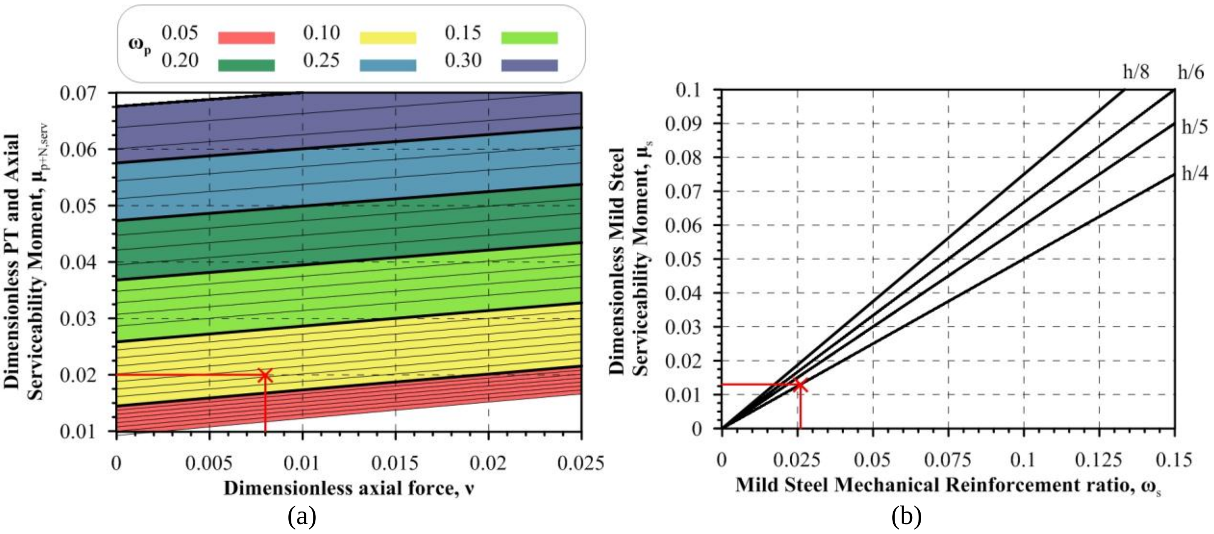


Figure 2.5. Design charts for (a) post-tensioning and (b) mild steel reinforcements

The mild steel reinforcement design chart (Fig. 2.5b) can be simplified throughout a linear expression of mechanical reinforcement ratio (Eqn. 2.3).

$$\mu_s = \left(1 - 2\frac{d_s}{h}\right)\omega_s \tag{2.3}$$

2.5. Design approach

The proposed preliminary design approach through the use of the two simplified design charts presented in Fig.2.4 can be broken down into the following steps:

1. Evaluate the seismic design actions in terms of moment and axial force from seismic and gravity loads respectively.
2. Define a re-centering ratio (λ) value (generally this is assumed to be 1.5) and evaluate dimensionless moment contributions

3. Evaluate the required post-tensioning steel and mild steel reinforcement for a given edge distance
4. Calculate the required area of post-tensioned and mild steel $A_p = \omega_p f_t bh/f_p$ and $A_s = \omega_s f_t bh/f_s$.
5. Perform a detailed section behaviour check according to a moment-rotation procedure as discussed in paragraph 2.1.

3. CASE STUDY BUILDING

According to the simplified design procedure shown in paragraph 2.5, the design steps of case study building shown in Fig. 3.1 are herein performed. It is assumed that shear post-tensioned rocking (single) walls are used for the seismic resistance in both directions. Internal gravity frames are used in the transverse direction, with timber-concrete composite floors spanning 8 m onto them.
Hazard factor Z=0.3 (peak ground acceleration, new proposed hazard level for Christchurch), return period factor of 1.30 (50 years working life and importance level 3, corresponding to a return period of the design event of 1000 years), fault factor of 1 (no near field effects), a displacement reduction factor η of 0.76 (corresponding to equivalent viscous damping of 10%) and soil type C were assumed.
A design drift level of 1.2% was considered.

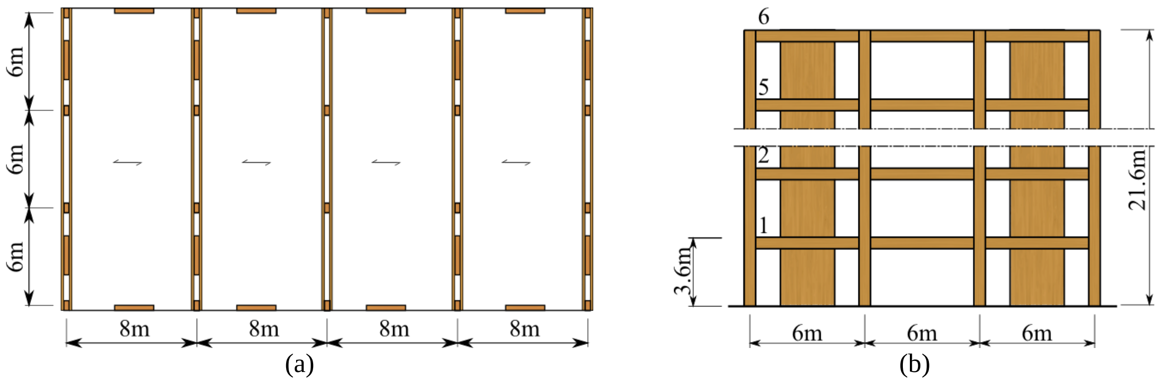


Figure 3.1. Case study (a) plan view and (b) transverse section

Step 1- Seismic design actions and gravity loads
The building's total self-weight is 192 tonnes, and, assuming live load for office buildings of $3kN/m^2$, the seismic weight for each floor is 244 tonnes (192t for the roof).
Following a Direct Displacement-Based Design Procedure (DDBD) (Priestley *et al.*, 2007) for the seismic design of the post-tensioned walls, the structure is converted to a Single Degree of Freedom (SDOF) equivalent structure.
The equivalent SDOF system has equivalent mass (m_e) of 1139 tonnes and equivalent height (h_e) of 15.211m; resulting design displacement (Δ_d) of 183mm. The resulting equivalent period for the structure is T_{eff}=1.73s; therefore secant stiffness (k_e) of 15022kN/m is resulting, and base moment base shear are 41709kNm and 2742kN respectively.
A total number of eight post-tensioned walls are assumed in each direction, so the required base shear and overturning moment for each wall are 343kN and 5214kNm, respectively. Thus, applying a strength reduction factor of 0.85 on the moment capacity, the amplified shear and moment design-level demand are 406kN and 6134kN.Each wall has a tributary area for gravity loading of approximately 1/30 of the total floor area, so the axial load acting on the wall element is assumed to be 400kN.
Step 2 – Moment contributions and dimensionless parameters
At this stage the dimensionless parameters can be evaluated and the simplified design tools presented in previous paragraphs are used for a preliminary evaluation of the wall reinforcements.
Considering a re-centering ratio λ value of 1.5, resulting moment contributions according to Eqn. 2.1 are $M_s = 2454kNm$ and $M_p + M_N = 3680kNm$. For wall section standard dimensions of 315×3600mm, the following dimensionless parameters are $\nu = 0.008$, $\mu_s = 0.013$, $\mu_p + \mu_N = 0.020$ (from Eqn. 2.2).

Step 3 – Post-tensioning and mild steel reinforcement ratios

The preliminary evaluation through charts of Fig. 2.5 lead to mild steel (assumed edge distance is h/4) and post-tensioning reinforcement ratios of 0.026 and 0.10 respectively, and a post-tensioning initial stress of 50%f_{py}.

Step 4 – Reinforcement areas

The corresponding mild steel and post-tensioning reinforcement areas are A_s = 3426mm^2 and A_p = 6111mm^2. Therefore, 2D50mm post-tensioning rebars and (6+6)D28mm (top + bottom) mild steel bars are needed.

Step 5 – Detailed checks

The detailed checks through a moment-rotation analysis are now performed to validate the preliminary design achieved above.

Some parameters which were not taken into account during the parametric study shall now be evaluated.

For this example the post-tensioning unbonded length is assumed to be equal to the height of the wall (26100mm), and the mild steel unbonded length is l'_{ub} = 450mm (h/8).

As shown in Fig. 3.2a, mild steel and post-tensioning moment contribution of 2500kNm and 3831kNm respectively are achieved, resulting in a total moment capacity of 6330kNm which satisfies the (amplified by 1/ϕ) demand (M = 6134kNm). Check on re-centering ratio shall be performed as well; for the design example this results in a value λ = 1.53.

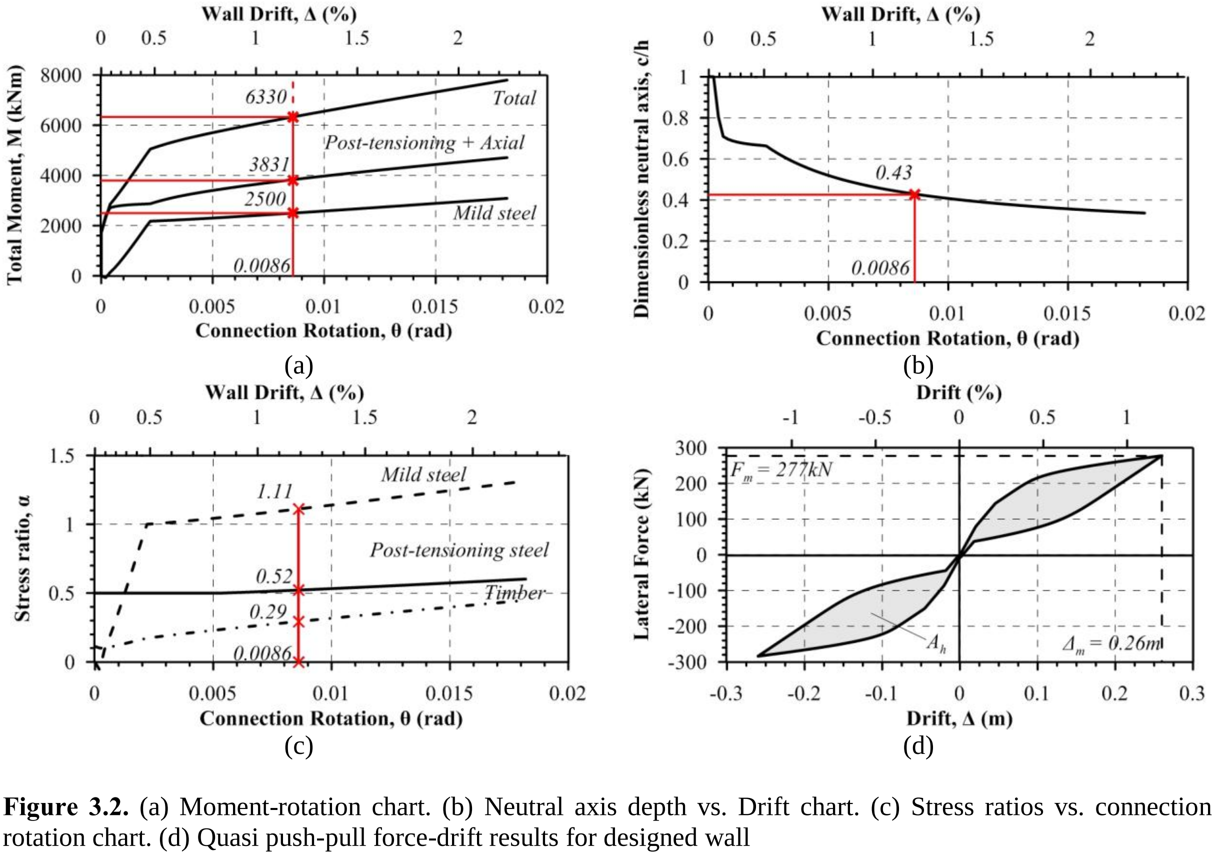


Figure 3.2. (a) Moment-rotation chart. (b) Neutral axis depth vs. Drift chart. (c) Stress ratios vs. connection rotation chart. (d) Quasi push-pull force-drift results for designed wall

The moment capacity at 1.2% wall drift, corresponding to 0.86% connection rotation, satisfies the moment demand. Neutral axis depth of 43% of the section height is resulting at 0.86% connection rotation; this means one of mild steel layers is in compression, and post-tensioning reinforcements are in tension. From the neutral axis depth and the connection rotation stress checks can be performed. Fig. 3.2c reports the stress ratio for each component; the parameter α in the chart represent the ratio of

either mild steel, post-tensioning steel or timber stresses over their respective yield stresses (see Table 2.1).
As shown in the chart at 1.2% drift (correspoding to 0.86% connection rotation, 0.34% is the elastic deformation of the wall) the post-tensioing and mild steel reinforcements reached respectively 52% and 110% of the yield stress, and timber is at 30% of its yield stress. The sectional checks are thus deemed to be satisfactory.
A push-pull analysis based on lateral force and lateral drift (Fig. 3.2b) is carried out, based on the moment-rotation behaviour of the section/connection (shown in Fig. 3.2a) in order to confirm the overall behaviour and evaluate the equivalent viscous damping (ξ_{eq}) of the system, given by the sum of elastic (ξ_{el}) and hysteretic damping (ξ_{hyst}). Previous research on seismic behaviour of timber structures have recommended values between 2% and 5% for the elastic damping (Filiatrault *et al.*, 2003, Christovasilis *et al.*, 2007, Pino *et al.*, 2010), value of 2% is herein conservatively adopted for post-tensioned timber walls (Newcombe, 2012). The elastic damping is then corrected considering the relationship shown below (ductility factor μ for the wall is 2.45, a = -0.43for a flag-shape hysteresis). Hysteretic damping is evaluated as area-based damping (Jacobsen, 1960) (See Eqn. 3.1).

$$\xi_{eq} = \kappa\xi_{el} + \xi_{hyst} = 0.68 \cdot 2\% + 9.9\% = 11.3\%; \kappa = \mu^{a} = 2.45^{-0.43} = 0.68;\ \xi_{hyst} = \frac{A_h}{2\pi F_m \Delta_m} = 9.9\% \qquad (3.1)$$

The equivalent viscous damping value of the structure is thus satisfactory (higher than 10% used to reduce the displacement design spectrum).

4.CONCLUSIONS

The paper presented a simplified design procedure for post-tensioned timber walls. The proposed design method, developed through an extended parametric study, is applied to material optimised standard sections.
The simplification of the design procedure allow for quick evaluation of post-tensioning and dissipater reinforcement. The procedure has been proved to be robust and reliable through a case study building.

In fact, detailed checks, based on the development of a more accurate moment-rotation section analysis proved to be consistent with the simplified methodology.
The procedure will be soon detailed for each dissipater types and generalized to other engineered timber materials, such as glue-lam and cross-lam.

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