Quasi static cyclic tests of 2/3 scale post-tensioned timber wall and column-wall-column (CWC) systems

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ABSTRACT: The paper presents the design and construction detailing of the quasi-static testing of two post-tensioned timber wall systems: a single (more traditional) wall system and a new configuration comprising of a column-wall-column coupled system (CWC). The latter allows avoiding displacement incompatibilities issues between the wall and the diaphragm by using the boundary columns as supports.

Different reinforcement configurations were taken into account for both the wall systems; the walls were subjected to different initial post-tensioning stress levels, and different dissipater options were considered: both internal and external replaceable mild steel tension-compression yield fuses, and U-shape Flexural Plates (UFPs) were used for the single wall and the CWC solutions respectively.

The experimental results showed the high-performance of both post-tensioned timber wall systems with negligible level of structural damage in the wall element and residual displacements and high level of dissipation.

1 INTRODUCTION

Post-tensioning low-damage technologies have been first developed in the late 1990s as the 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, 1991).

The extension of post-tensioning techniques to timber elements brought to new structural systems, referred to as Pres-Lam. Pres-Lam systems, consist of large timber structural frames or walls made of Laminated Veneer Lumber (Palermo *et al.*, 2006) or any other engineered timber (Smith *et al.*, 2013). In particular post-tensioned walls proved efficient in absorbing seismic action. Such walls are rocking timber elements where self-centering contribution is provided by the post-tensioning unbonded tendons, while energy dissipation is supplied by either internal or external mild steel dissipaters.

Small-scale specimens of single wall subassemblies were tested with internal and external dissipaters (Palermo *et al.*, 2006; Smith *et al.*, 2007); moreover, coupled walls with U-shape Flexural Plates (UFPs) dissipaters were tested by Iqbal *et al.* (2007). The experimental results showed the system can provide high levels of hysteretic damping and excellent re-centering, and virtually no damage is observed in the structural members.

Following the extensive research on the Pres-Lam technology supported by the Structural Timber Innovation Company (STIC), few post-tensioned timber buildings were recently constructed in New Zealand. The Nelson and Marlborough Institute of Technology in Nelson was the first post-tensioned timber building constructed worldwide; it is a three storey timber building with coupled walls resisting the horizontal actions (Devereux *et al.*, 2011) which uses UFPs (Skinner *et al.*, 1974) as dissipative source between the walls. **Figure 1**b shows the Carterton Events Center (Carterton), a single story building with large timber trusses for carrying the gravity loads and single walls with internal dissipaters resisting the seismic loading (Palermo *et al.*, 2012).

Figure 1. (a) Nelson and Marlborough Institute of Technology (Devereux et al., 2011)and (b) Carterton Events Centre.

The vertical uplift generated by the gap opening at the base of the wall element can cause some displacement incompatibility issues when interacting with the diaphragm system as shown in **Figure 2**a. That vertical displacement can bring to damage in the diaphragm when the connection is not properly designed, thus influencing its capacity (Moroder *et al.*, 2014); moreover, the interaction of the two systems can increase the vertical load on the wall. The increased axial force amplifies the capacity of the wall system but reduces the energy dissipated of the system since the dissipaters might not be activated. The increased axial load can also cause global instability as well as higher local damage at the compression area.

To mitigate that issue, an alternative configuration (**Figure 2**b), referred to as Column-Wall-Column (CWC), was proposed and tested. The new solution comprises of a single wall as the main resisting system; boundary columns provide the support to the diaphragm drag beams and are coupled using U-shaped Flexural Plates (UFPs), also providing energy dissipation to the system. A similar precast concrete solution was proposed and tested by (Henry *et al.*, 2012).

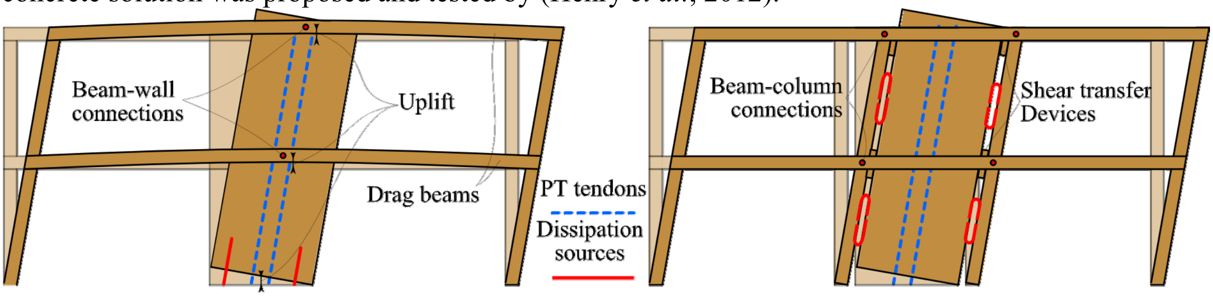

Figure 2. (a) Displacement incompatibilities issue for single wall; (b) Column-wall-column system.

This paper shows the seismic design, construction detailing and experimental results of the 2/3 scale testing of post-tensioned timber walls. Tests results are discussed based on experimental observations as well as in terms of area-based hysteretic viscous damping.

2 DESIGN AND DETAILING OF THE WALL SPECIMEN

2.1 Displacement-Based seismic Design

The prototype case study building is a three storey building with two suspended floors and a lightweight timber penthouse on third floor (modified from (Structural Timber Innovation Company (STIC), 2013a)). The building has an approximate plan of 32m in the longitudinal direction and 18m in the transverse direction with a floor area of approximately 600m^2 per floor. The ground floor level is assumed to be used for retail purposes, level 2 as office space, and level 3 will have a lightweight timber penthouse with residential type loadings.

The building was designed using the Christchurch CBD return period factor and a soil type C according to NZS1170.5 (Standards New Zealand, 2004). For design purposes, the seismic mass of the

light-weight top floor is concentrated at the second level; an equal storey seismic weight of 3046kN (762kN per wall) is resulting.

The multi-degree of freedom structure is converted into a single-degree of freedom (SDOF) equivalent system according to a Displacement-Based Design (DBD) procedure (Priestley *et al.*, 2007), and the design base shear and moment are 329kN and 1645kNm per wall respectively.

Due to height and space constraints the full-scale prototype of the wall was tested on a 2/3 scale. The timber section of the specimen was 0.189×1.57m and made up of standard 63mm thick Laminated Veneer Lumber (LVL) profiles chosen to optimize the section.

The specimen design is performed using a scaled shear/moment demand:

$$V_{2/3} = \lambda^2 V_b = (2/3)^2 \cdot 329kN = 146kN, M_{2/3} = \lambda^3 M_b = 487kNm \quad (1)$$

To resist the scaled design loads, the single wall specimen had two 32mm post-tensioning bars stressed to a total initial post-tensioning force of 400kN (30%f_{py},24% f_{pu}), and 8D14mm mild steel dissipaters. An equivalent layout can be obtained for the CWC specimen using 8 12mm thick, 130mm wide UFPs with radius of 40mm.

2.2 Testing specimens detailing

The two systems and testing specimens are shown in **Figure 3**. The benchmark option is a single wall system where the dissipation is concentrated at the base connection via tension-compression yielding fuse-type dissipaters. Riveted connections (see §2.2.3) provide the attachment to the timber wall element and allow the quick replacement of the dissipaters.

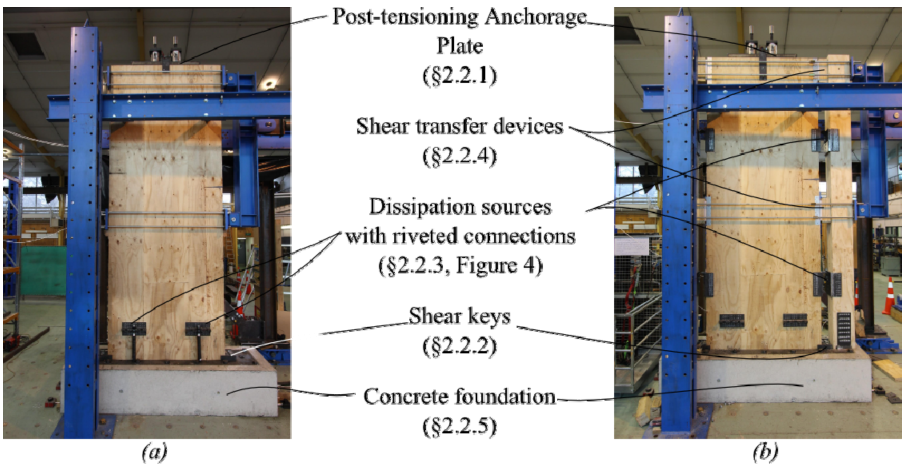

Figure 3. (a) Single wall and (b) CWC systems test specimens

A possible issue with a traditional single wall system regardless being a rocking or "monolithic" type is that it can strongly interact with the diaphragm. In rocking elements an uplift displacement generates from the base gap opening. Such vertical displacement can damage the diaphragm by inducing forces not considered in the design, on the other side the presence of the diaphragm can act as a restraint and increase the vertical force acting on the wall. Such increase in the vertical force would change the response of the system, increasing the moment capacity and re-centering contribution of the wall, thus lowering the hysteretic damping; furthermore, the increased axial stress at the compression area can bring to unexpected damage.

Although the wall-diaphragm interaction can cause displacement incompatibilities issues, the connections can be properly designed to account for those effects; more details on diaphragm to wall connection can be found in (Moroder *et al.*, 2014).

An alternative solution, referred to as Column-Wall-Column (CWC) system (**Figure 3**b), was proposed and tested to mitigate this potential issue. Two boundary columns, which are not affected by uplifting, are placed on the sides of the wall and provide the support for the diaphragm and the

dissipaters. The shear is transferred using some sliding devices as discussed in §2.2.4.

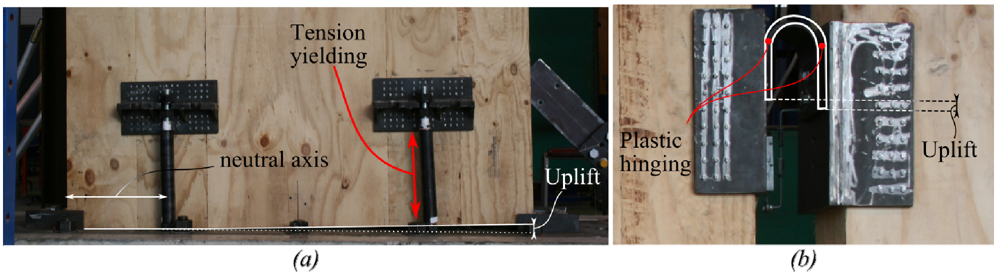

Figure 4. Dissipation mechanisms. (a) Single wall and fuse-type dissipaters; (b) CWC system with UFPs

2.2.1 *Wall optimized section and post-tensioning anchorage*

The timber section (**Figure 5**a) of the specimen was 0.189×1.57m and made up of standard 63mm thick Laminated Veneer Lumber (LVL) profiles chosen to optimize the section. The voids created for the post-tensioning bars are reduced to a minimum to have the biggest possible bearing area for the post-tensioning anchorage at the top of the wall.

Figure 5b shows the wall before erection with the detail of the anchorage plate. A Grade300 60mm thick plate was used.

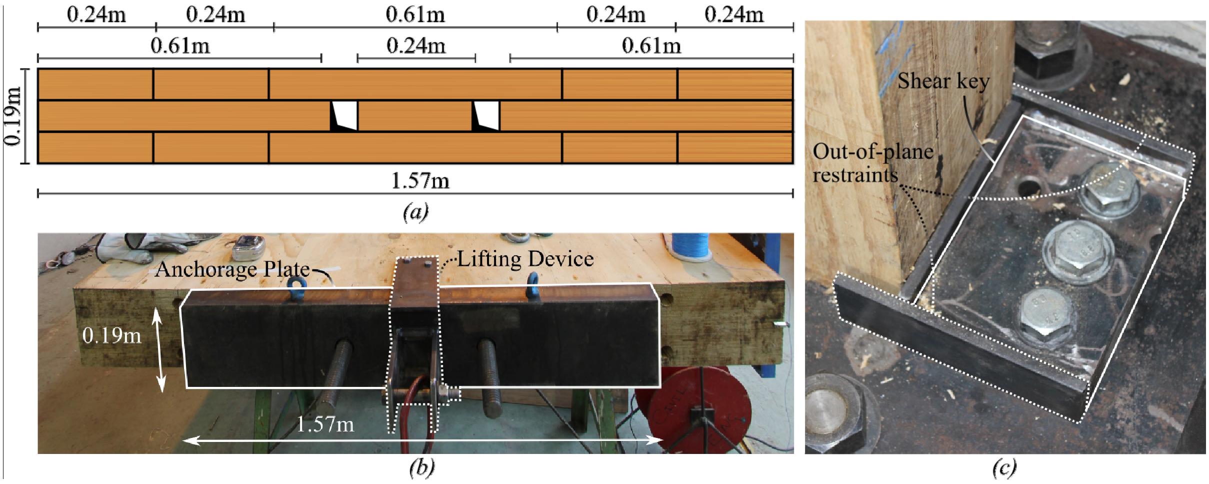

Figure 5. (a) Wall section. (b) Post-tensioning plate detail; (c) shear key

2.2.2 *Shear key*

Although a large amount of friction is developed at the base of the wall due to the high compressive forces, some shear keys were designed to prevent lateral displacement longitudinally to the wall; moreover, the out-of-plane movement of the wall base can occur. To provide both of these restraints, 40mm thick plates were used to prevent sliding, and thinner 6mm plates were welded on the sides to avoid out-of-plane displacements. The detail of the shear key is shown in **Figure 5**c.

2.2.3 *Dissipater connections*

When using external dissipation devices, the dissipater-to-wall connection must be detailed carefully in order to provide the required strength as well as to limit the elastic deflection coming from the connection flexibility. In fact, the latter can bring to the activation of the dissipaters at a higher drifts reducing the energy dissipation. The common practice in timber engineering is the use of metallic fasteners such as screws, bolts or timber rivets. The latter represent a suitable solution for this type of application since they allow the creation of a connection with high stiffness with no significant overdesign (Structural Timber Innovation Company (STIC), 2013b). A detailed picture of the connection is shown in **Figure 6**.

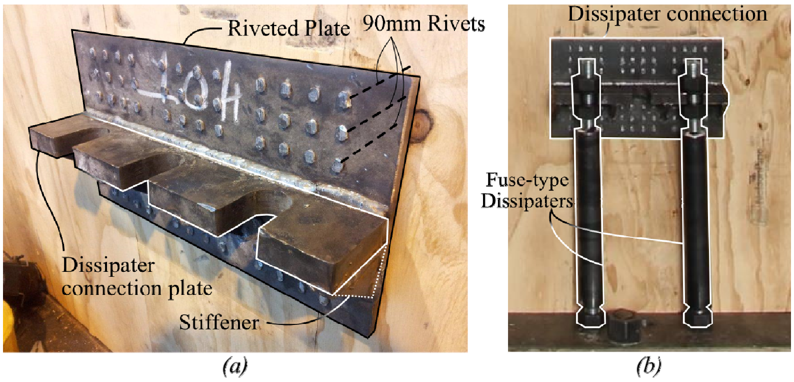

Figure 6. Fuse-type dissipater connection: (a) detail of the connection; (b) final setup.

For the column-wall-column (CWC) option the UFP connection was designed using a similar solution. The M16 threaded holes for the UFPs connection were drilled on a 20mm thick steel plate as shown in **Figure 7**. Two thinner side plates were welded to the connection plate to provide the connection to the timber element.

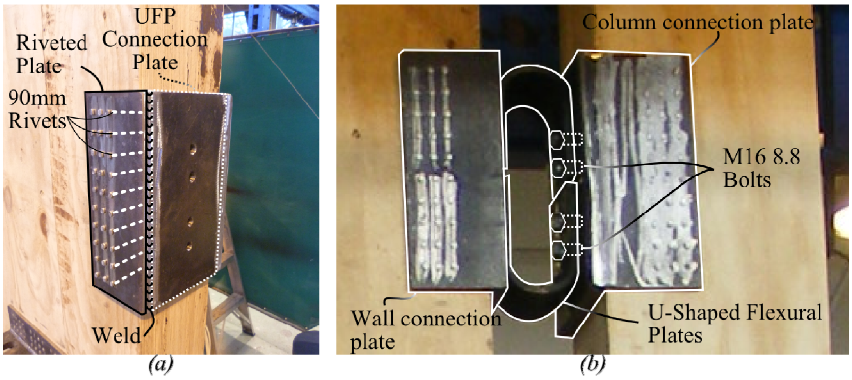

Figure 7. UFP connection: (a) detail of the welded connection and riveted plates; (b) detail of the final setup

2.2.4 *Shear transfer devices*

To transfer the shear forces between the boundary columns and the wall, a sliding connection was designed and detailed to minimize the contribution due to friction. The reason of the design choice was to reduce additional dissipative contributions, which could lead to residual displacements after testing. The connection sketch is shown in **Figure 8**. The connection is a Laminated Veneer Lumber (LVL) element connected to the column through inclined self-tapping screws. A 20mm thick High-Density Polyethylene sheet was screwed to the block surface, and stainless steel thin plates were fixed to the side of the wall.

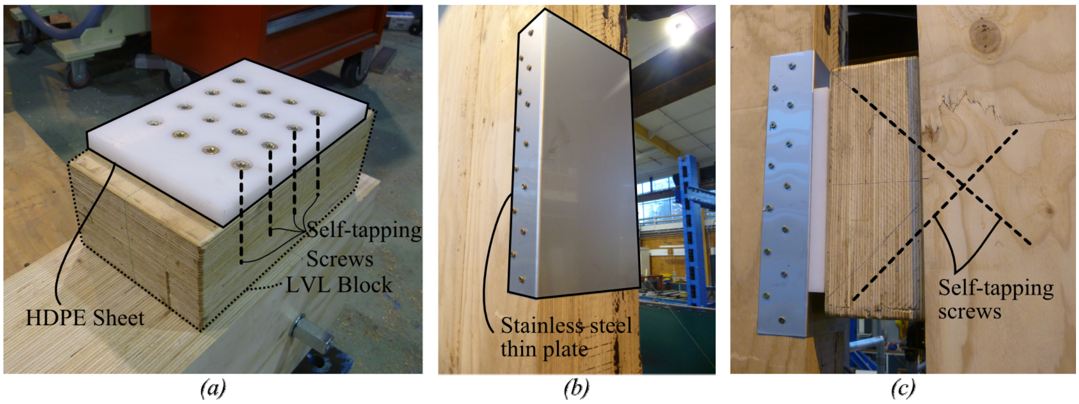

Figure 8. Shear transfer device. (a) LVL and HDPE block on the column; (b) Stainless steel sheet on the wall side; (c) the finished setup.

2.2.5 *Foundation details*

A reinforced concrete foundation was used for supporting the wall base. The concrete block was 0.53m deep, 0.76m wide and 2.66m long. Several ducts were created in the foundation to allow the connection to the strong floor with anchoring bolts. To accommodate the post-tensioning anchorage devices into the foundation, a pocket was created underneath the concrete element. A 32mm steel plate was positioned on the top of the foundation to create an even surface for the wall base as well as the necessary threaded holes for connections. Different holes were drilled to connect the shear keys as well as to provide flexible positioning of the dissipaters (either internal or external).

3 TESTING METHODOLOGY

3.1 Test setup

The test setup is shown in **Figure 9**a. The wall was loaded with a triangular distribution of forces accordingly to the case study design which assumed the top floor mass concentrated at the second level. The load pattern was achieved using a 2m deep distribution beam which was linked to the wall and the actuator through steel hinges. The reaction frame consisted of a built-up column made of a double 300PFC profile, further stiffened along the length by welded plates; lateral stability was provided by two struts (SHS100 and SHSH200 steel profiles). The load was applied by a hydraulic jack with a 1000kN load cell at a height of 3.7m from the strong floor (3.2m from the bottom of the wall).

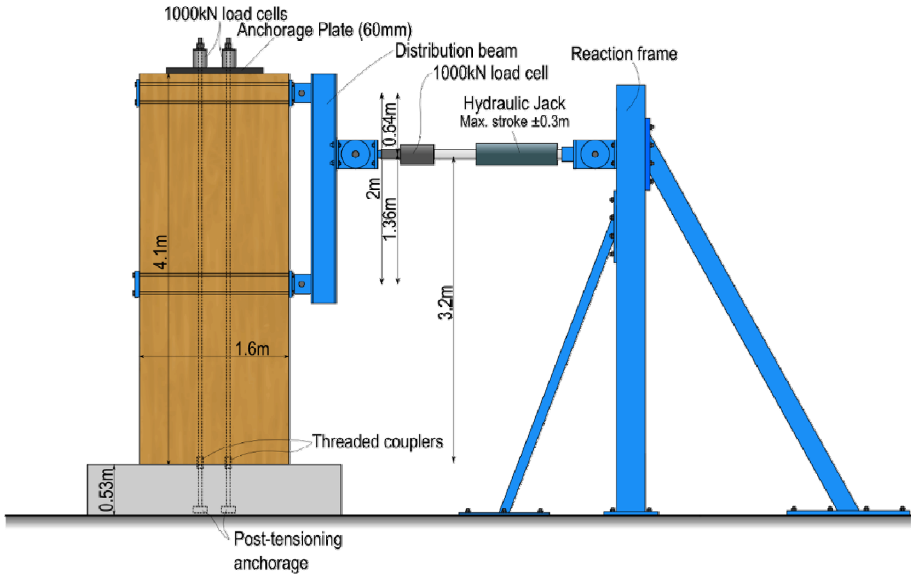

Figure 9. Experimental test setup

3.2 Testing protocol

The quasi-static displacement protocol consisted of subsequent displacement-controlled cycles according to ACI ITG-5.1-07 (ACI Innovation Task Group 5, 2008) protocol: the maximum displacement of the first three amplitudes does not exceed 60% of the design displacement (Δ_d = 0.040m); the maximum displacement of the subsequent cycle amplitudes is between 1.25 and 1.5 times the previous maximum displacement. Lateral drifts of 0.3%, 0.45%, 0.6%, 1.0%, 1.5% and 2.0% were imposed. The maximum drift imposed corresponds to the MCE displacement.

3.3 Testing schedule

The testing schedule in Table 1 was developed starting from the benchmark design presented above. Tests on the single wall specimens ("S" specimens in Table 1) were first performed, varying the reinforcement configuration in terms of post-tensioning initial force and mild steel reinforcement

layout. Several tests with post-tensioned only configurations were carried out, increasing the post-tensioning forces from 200kN to a maximum of 600kN. Hybrid specimens (i.e. with dissipation) were tested with different number of dissipaters to assess the influence of the re-centering ratio. For the single wall internal epoxied bars as well as fuse-type external dissipaters (Sarti *et al.*, 2013) were used.

The alternative Column-Wall-Coulmn configuration was also proposed and tested. The new configuration relies upon the use of U-shaped Flexural Plates (UFPs) as dissipation devices (Skinner *et al.*, 1974; Baird *et al.*, 2014) between the central wall and two adjacent columns.

Table 1. Post-tensioned walls testing schedule.

Test ID	PT Initial	Dissipaters	β	Test ID	PT Initial	Dissipaters	β
S200	200kN	n/a	1.00	S600-4i	600kN	4D14mm (internal)	0.80
S400	400kN	n/a	1.00	CWC400	400kN	n/a	1.00
S600	600kN	n/a	1.00	CWC600	600kN	n/a	1.00
S400-8	400kN	8D14mm	0.70	CWC600-4	600kN	4UFPs	0.70
S400-4	400kN	4D14mm	0.75	CWC400-4	400kN	4UFps	0.75
S600-4	600kN	4D14mm	0.80	CWC400-8	400kN	8UFPs	0.80

NOTE $\beta = M_{pt}/M_{tot}$. β = re-centering ratio; M_{pt} = post-tensioning moment contribution, M_{tot} = total moment

4 RESULTS

Test results on the post-tensioned only single wall specimens highlighted the expected multi-linear elastic behaviour (see **Figure 10**a, red line). Some minor plastic behaviour was observed for the S600 specimen (stress values in Table 2) which had the highest initial post-tensioning force level. Minor plasticization at the wall-to-foundation interface leads to a rather negligible energy dissipation. Although some plastic behaviour is present, neither residual deformation nor significant damage to the element were observed.

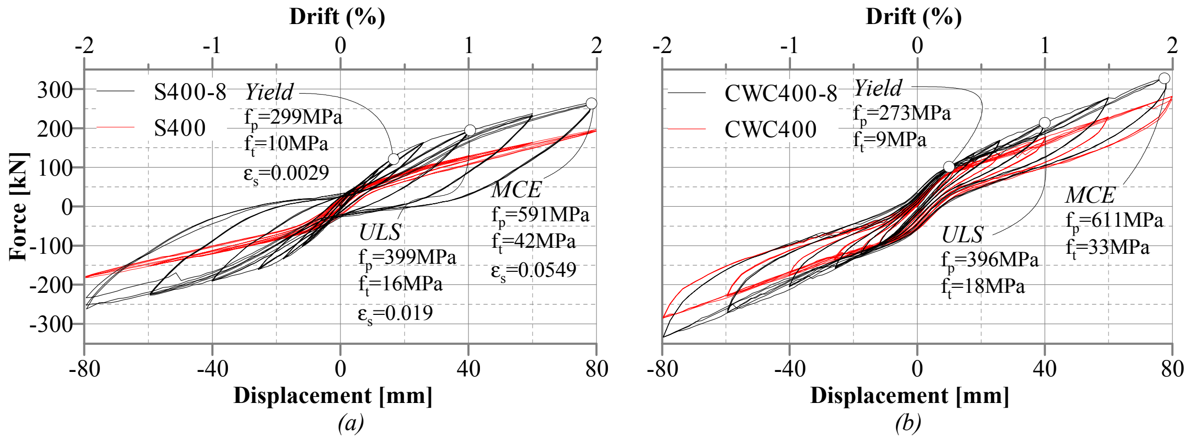

Figure 10. Experimental results. (a) Single wall; (b) CWC system (for notation refer to Table 2).

Some post-tensioning losses occurred in the post-tensioned only tests. The losses are due to two main factors: minor plastic deformations occur on the surface of the bearing plates due to highly concentrated stresses on the reduced area of the washer; in addition, the yielding stress (835MPa), according to the manufacturer's specifications is defined as 0.1% proof stress; therefore, small plastic deformations are allowed for. At the end of the test such losses were 42kN, 40kN and 67kN for 200kN (15%f_{py}), 400kN (30%f_{py}) and 600kN (45%f_{py}) initial post-tensioning force respectively.

The benchmark solution (S400-8) performance (see **Figure 10**a) shows a significant amount of dissipation, yet full re-centering was not achieved. The residual deformations are due to over-strength of the dissipaters as well as isotropic hardening in compression. Those effects were not accounted for in the preliminary design (i.e. 70% post-tensioning and 30% dissipative contributions). The same capacity was fulfilled by the S600-4 specimen, and full re-centering was achieved with a lower amount of dissipated energy.

The experimental results of the post-tensioned only Column-Wall-Column (CWC) specimens (**Figure 10**b, red line) highlighted an increased capacity when compared to the equivalent single wall solutions due to the stiffening of the two boundary elements; moreover, the timber blocks transferring the shear force from the column to the wall dissipate energy by friction, adding capacity. The increased moment capacity due to the combination of the two effects is approximately 20% of the total capacity.

Table 2. Experimental data results, summary of maximum values.

	Yield					***ULS***						***MCE***					
ID	**f_p (MPa)**	**γ_p (-)**	**f_t (MPa)**	**γ_t (-)**	**ε_s (-)**	**f_p (MPa)**	**γ_p (-)**	**f_t (MPa)**	**γ_t (-)**	**ε_s (-)**	**ξ_{hyst} (%)**	**f_p (MPa)**	**γ_p (-)**	**f_t (MPa)**	**γ_t (-)**	**ε_s (-)**	**ξ_{hyst} (%)**
S200						339	0.40	18	0.40			562	0.67	36	0.79		
S400						437	0.52	20	0.52			646	0.77	35	0.79		
S600						539	0.65	35	0.80			747	0.89	45	1.00		
S400-8	299	0.36	10	0.22	0.0029	399	0.48	16	0.35	0.0190	8.4	591	0.71	42	0.92	0.0549	10.8
S400-4	292	0.35	8	0.17	0.0032	427	0.51	19	0.42	0.0256	6.9	634	0.76	41	0.90	0.0611	8.6
S600-4	405	0.49	15	0.33	0.0031	522	0.63	20	0.45	0.0227	5.7	715	0.86	35	0.77	0.0579	8.0
S600-4i	396	0.47	14	0.31	0.0278	526	0.63	25	0.56	0.0232	6.9	755	0.90	44	0.99	n/a*	n/a*
CWC400						440	0.53	29	0.65		3.9	651	0.78	42	0.94		3.8
CWC600						523	0.63	27	0.59		4.7	735	0.88	44	0.97		4.4
CWC400-8	273	0.33	9	0.19		396	0.47	18	0.40		6.3	611	0.73	33	0.73		7.8
CWC400-4	274	0.33	8	0.18		393	0.47	18	0.39		6.5	603	0.72	32	0.72		7.2
CWC600-4	399	0.48	11	0.24		514	0.61	23	0.50		6.2	722	0.86	37	0.83		7.4

NOTATION: f_p = maximum post-tensioning stress; f_t =maximum timber stress; ε_s = maximum dissipater strain; ξ_{hyst} = area-based hysteretic damping; $\gamma_p = f_p/f_{py}$ (f_{py} post-tensioning steel strength); $\gamma_t = f_t/f_{ty}$ (f_{ty} timber strength)
* Dissipaters failed at 1.5% drift, at MCE drifts the specimen behaved as a post-tensioned only solution

Similarly to post-tensioned only solutions, the hybrid CWC specimens (**Figure 10**b, red line) fulfilled increased capacity due to the combination of the stiffening effect of the boundary elements and the energy dissipation of the friction elements. In particular, the stiffening effect of the side columns, brings to a reduction of the energy dissipated by the U-shaped Flexural Plates (UFPs) in terms of equivalent viscous damping (see Table 2).

Stress and strain values (Table 2) at design level as well as at MCE drifts are within the material strength limits and performance limits suggested in (Structural Timber Innovation Company (STIC), 2013a). The suggested stress and strain limits are: (a) ULS – ε_s = 3-4%, $f_t = 0.5f_{ty}$ $f_p = 0.7f_{py}$; (b) MCE – ε_s = 5-6%, $f_p = 0.9f_{py}$.

Lower timber stresses were observed in the CWC system, potentially allowing reducing the wall dimensions and optimizing the section.

5 CONCLUSIONS

The design, construction detailing and experimental quasi-static testing of several post-tensioned timber specimens was presented in the paper, and discussion on experimental results was provided. The experimental campaign comprised two different systems: single wall and column-wall-column system; both post-tensioned only (elastic) and hybrid tests were performed.

The post-tensioned only test results showed the typical bilinear elastic behaviour, yet minor plastic behaviour was observed for high levels of initial post-tensioning force in the single wall specimens. In the CWC system a similar behaviour was observed, and additional dissipation contribution was provided by the friction developed in the shear transfer blocks. Hybrid specimens were also tested and both the systems proposed are capable of providing significant amount of dissipation. Detailed experimental data on materials stress and strain levels showed acceptable value also considering design guidelines proposed in (Structural Timber Innovation Company (STIC), 2013a). The technical design details adopted and already implemented in real case study buildings proved to be structurally robust and no unexpected failures occurred.

6 ACKNOWLEDGEMENTS

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