SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

EVALUATION OF THE SEISMIC PERFORMANCE FACTORS OF POST-TENSIONED TIMBER WALL SYSTEMS

Francesco SARTI[1], Alessandro PALERMO[2], Stefano PAMPANIN[3], Jeffrey BERMAN[4]

ABSTRACT

Low-damage seismic-resistant post-tensioning technologies were first developed during the PREcast Seismic Structural Systems program, coordinated by the University of California San Diego. Different connections were developed and tested as part of the research program, and the most stable solution was the hybrid connection, which provides a combination of re-centering and dissipative contributions. The hybrid connection was later extended to Laminated Veneer Lumber Elements (LVL) and referred to as Pres-Lam (Prestressed Laminated) system. As part of a broader experimental campaign on frame and walls systems, several experimental tests were carried out on small-scale specimens of post-tensioned single walls and on coupled walls systems. More recently 2/3 scale quasi-static tests were performed on different wall configurations.

The paper shows the evaulation of the seismic performance factors of post-tensioned timber wall systems, carried out according to the FEMA P695 procedure. The latter utilizes nonlinear analysis techniques, and explicitly considers uncertainties in ground motion, modelling, design, and test data. The technical approach is a combination of traditional code concepts, advanced nonlinear dynamic analyses, and risk-based assessment techniques.

A set of archetype buildings were developed to characterize the behaviour of the system. Several parameters were accounted for, such as the building height, lateral load resisting system, magnitude of the gravity loads and seismic design category. The system archetypes were represented by numerical models developed to simulate the full range of behavioural aspects of the system. Non-linear quasi-static and dynamic analyses were carried out to determine the system over-strength factors and median collapse capacity of the buildings. The system performance was then assessed by computing the Collapse Margin Ratio (CMR) defined as the ratio of the median collapse (S_{CT}) and MCE (S_{MT}) spectral accelerations.Once the non-linear analysis results confirmed the CMR values were within acceptable values, the trial value of the seismic response modification, R, was confirmed, and the system seismic performance factors were evaluated.

[1] PhD Candidate, Civil and Natural Resources Engineering, University of Canterbury, New Zealand, francesco.sarti@pg.canterbury.ac.nz

[2] Senior Lecturer, Civil and Natural Resources Engineering, University of Canterbury, New Zealand, alessandro.palermo@canterbury.ac.nz

[3] Professor, Civil and Natural Resources Engineering, University of Canterbury, New Zealand, alessandro.palermo@canterbury.ac.nz

[4] Associate Professor, Civil and Environmental Engineering, University of Washington, Seattle, Washington, United States of America, jwberman@uw.edu

INTRODUCTION

Post-tensioning rocking systems were first investigated during the PREcast Seismic Structural Systems program (PRESSS), coordinated by the University of California San Diego in the 1990s (Priestley, 1991) Among different solutions tested in the last phase of the PRESSS program, the hybrid connection proved capable of providing excellent seismic behaviour. The system provides a combination of re-centering and dissipation, while limiting the damage to the structural element and the residual displacements after the event.

The concept of hybrid system has subsequently been extended to steel structures (Christopoulos et al., 2002), proving to be material-independent, and, in more recent year, to timber (engineered wood) structural systems (Palermo et al., 2005). 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.

The Pres-Lam walls concept

In Pres-Lam walls, the structural element consists of a large timber panel, generally Laminated Veneer Lumber (LVL). The un-bonded post-tensioning reinforcements are usually high-strength steel bars or strands and re-center the system, while alternative devices can be used to provide energy dissipation. In single walls systems, the dissipative reinforcement is concentreated at the base connection of the wall and consists of either internal or external (repleacable) tension-compression yield mild steel dissipaters.

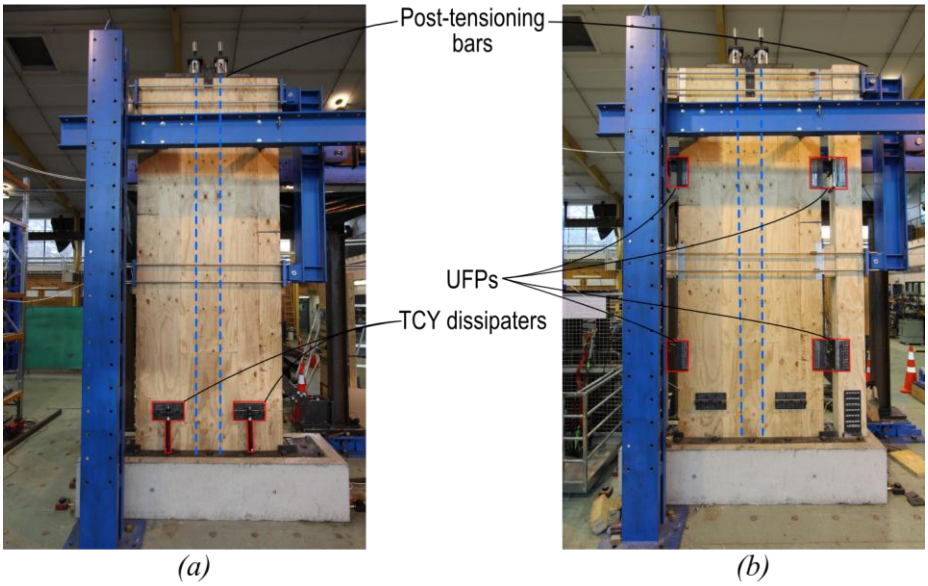

Figure 1. Pres-Lam walls. (a) Single wall; (b) Column-wall-column coupled system .

Coupled walls options are available as well. Researchers have been carrying out experimental tests on coupled walls (Iqbal et al., 2007) as well as column-wall-column coupled systems (Sarti et al., 2014). In both solutions, coupling and dissipation are provided by mild steel U-shape Flexural Plates (UFPs).

The behaviour of the post-tensioned rocking connection can be represented by the combination of a multi-linear elastic and a bi-linear hystereses as shown in Figure 2. The combination of the two hysteretic rules generates the “flag-shape” hysteresis. The shape of this relationship is governed by the re-centering ratio, β, defined as:

$$\beta = \frac{M_{pt}}{M_{tot}} \tag{1}$$

Where M_{pt} is the post-tensioning moment contribution to the total moment M_{tot}.

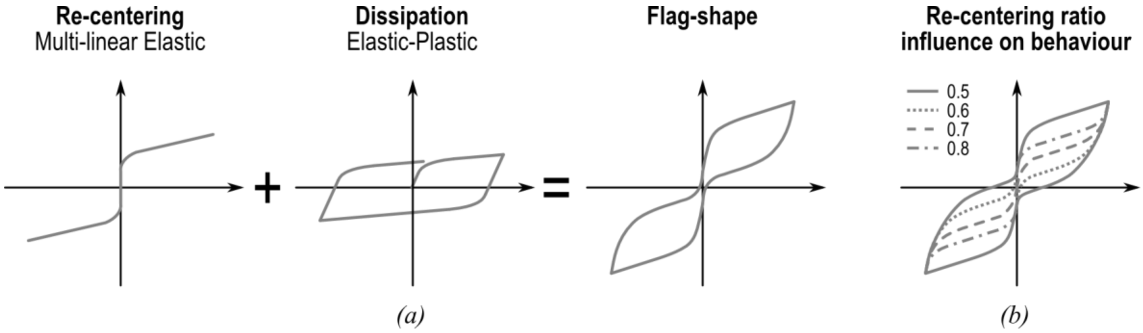

Figure 2. Qualitative moment-rotation behaviour of post-tensioned rocking sections. (a) Contributions and (b) influence of re-centering ratio

Figure 2c qualitatively shows the influence of this parameter on the hysteric behaviour of the connection. For $\beta=1.0$, the connection behaves following a multi-linear elastic (post-tensioned only) relationship, with no or negligible hysteretic dissipation; instead, for $\beta = 0$ the connection is a mild steel-only option with substantial hysteretic dissipation but also significant residual displacement. A minimum value of 0.6 is generaly assumed, ensuring acceptable levels of dissipation and negligible residual displacement.

OVERVIEW OF THE FEMA P695 PROCEDURE

The determination of seismic performance factors of post-tensioned timber systems was carried out according to the FEMA P695 procedure (ATC 2009). The latter utilizes nonlinear analysis techniques, and explicitly considers uncertainties in ground motion, modelling, design, and test data. The technical approach is a combination of traditional code concepts, advanced nonlinear dynamic analyses, and risk-based assessment techniques (Applied Technology Council for the Federal Emergency Management Agency, 2009). The flow chart in Figure 3 summarizes the general framework of the procedure.

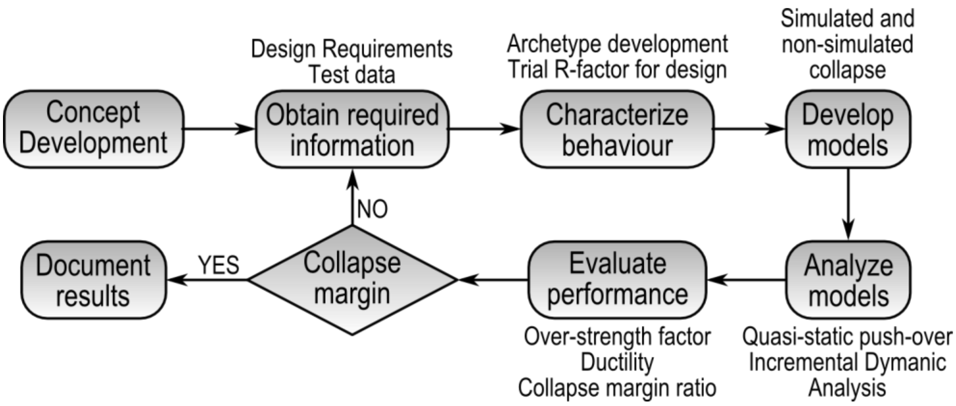

Figure 3. FEMA P695 Methodology framework (modified from ATC (2009)).

REQUIRED INFORMATION

Design requirements

The seismic design requirements of ASCE/SEI 7-10 (2010) are adopted for the determination of the seismic loads for each index archetype.

In addition to the seismic design requirements of ASCE 7-10, some ad hoc system requirements are suggested in this study, also according to the STIC Design Guidelines (Structural Timber Innovation Company (STIC), 2013).

The general performance-based design philosophy of post-tensioned timber systems can be qualitatively summarized by Figure 4. The general hierarchy of strength and sequence of events would be (Structural Timber Innovation Company (STIC), 2013):

1. Yield of the non-prestressed reinforcement (or dissipaters)
2. Yield of the timber at the rocking interface either in the column or the beam
3. Finally yielding of the post-tensioning reinforcement.

Such design philosophy allows the system to:

1. withstand small and frequent events with no significant damage
2. provide the required hysteretic damping under a design level earthquake
3. prevent structural collapse when subjected to a Maximum Credible Event (MCE)

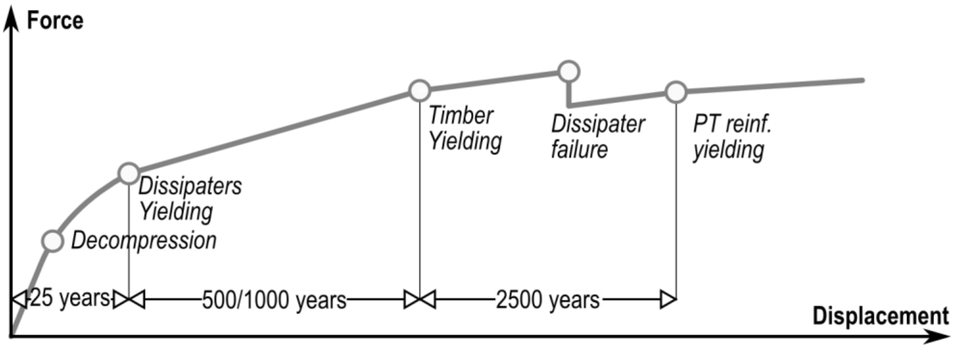

Figure 4. Qualitative push-over curve and performance limits.

Design provisions were suggested in (Structural Timber Innovation Company (STIC), 2013) for seismic design according to the New Zealand seismic design standard (Standards New Zealand, 2004), yet the seismic design methodology differs significantly from ASCE 7-10; therefore, material limit states are proposed to be implemented in the seismic design methodology of ASCE 7-10.

The proposed design requirements include suggested values of re-centering ratio, strain limits in the dissipaters at MCE level and initial timber stress:

- The seismic resistant element is elastically designed at design level forces ($V = 2V_{MCE}/3R$) according to ASCE 7-10 (American Society of Civil Engineers et al., 2010).
- Initial timber stress shall not exceed 10% of the timber compressive strength.
- The maximum strain in the tension-compression yield dissipaters shall be in the range of 3-4% at MCE level.
- Values of re-centering ratio (β) at design level shall be approximately 0.6 (60% re-centering contribution and 40% dissipative contribution).

Experimental data

Analytical modelling of post-tensioned rocking systems was first proposed by Pampanin et al. (2001) and modified by Palermo (2004). More recently the analytical procedure was adapted to timber post-tensioned elements (Newcombe et al., 2008). The model relies on an iterative moment-rotation analysis based on an equivalent monolithic solution. The analogy is required when dealing with post-tensioning rocking sections since the use of post-tensioning unbonded reinforcement (as well as, in case, to partially unbonded non-prestressed reinforcements) does not allow the consideration of the Euler-Bernoulli assumption of plane section and a strain compatibility relationship is unavailable at a section level but needs to be assumed at a member level not (Pampanin et al., 2001). The model is capable of accurately predicting the monotonic behaviour of the system and can be used in the design phase; however, for predicting and modelling the cyclic response of the system, comprehensive experimental data is required.

Experimental tests on coupled wall systems were carried out by Iqbal et al. (2007) experimentally investigated small scale (1/3) coupled wall systems; larger scale (2/3) single walls and the Column-Wall-Column (CWC) systems were carried out by Sarti et al. (2014). For each system, several elastic tests with increasing post-tensioning loads were performed and different dissipation options were taken into account.

For the single wall configuration tension-compression yielding mild steel devices (fuse-type dissipaters) are used. The device is made of a mild steel bar machined down to a reduced diameter to concentrate yielding in the central part of the dissipater. Threaded ends provide the dissipater connection to the structural element. To avoid buckling, the fuse area is encased in a steel tube which is filled with either grout or epoxy. Few component testing was carried out by Marriott et al. (2009) and Amaris (2010); more recently, a more extensive experimental and nuemerical study on this type of dissipation device was carried out by Sarti et al. (2013).

In CWC and coupled walls systems the structural elements are connected with U-shaped flexural plates. Those devices make use of the rolling (bending) of flat mild steel plates. The dissipater operates between adjacent surfaces whose relative motion is directed parallel to each other. Several tests were performed by Kelly et al. (1974), Iqbal (2007) and Baird et al. (2014). Analytical models were first developed by Kelly et al. (1974) and refined by Baird et al. (2014).

ARCHETYPE DEVELOPMENT

The development of the archetype buildings took into consideration the case study building with the plan view shown in Figure 5.

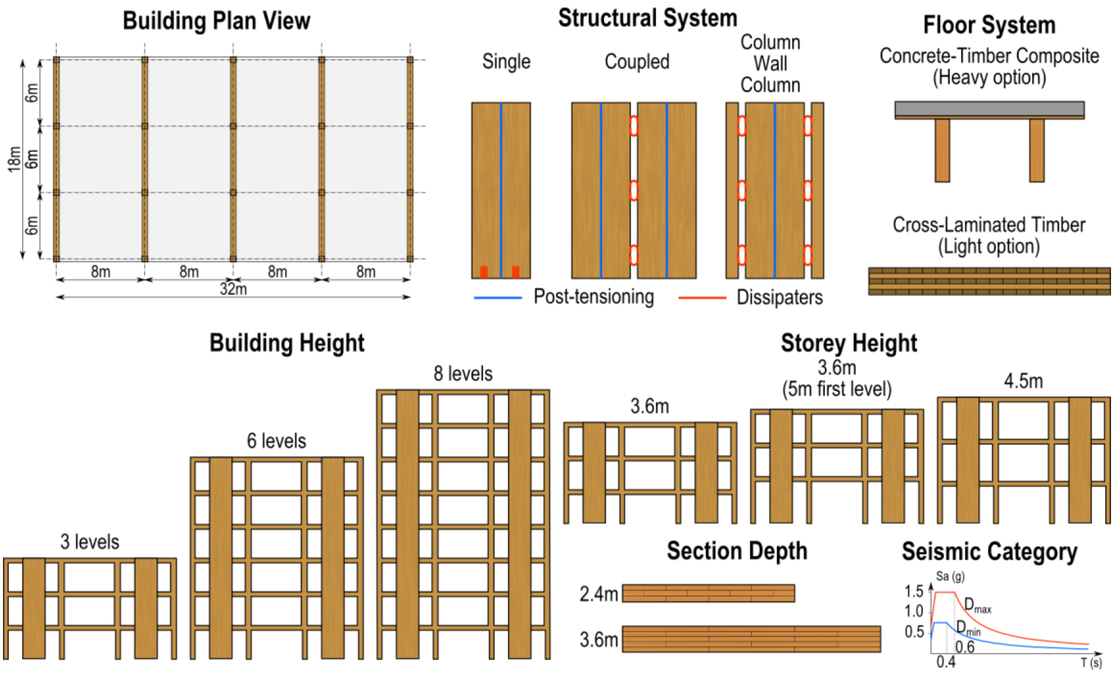

Figure 5. Archetype development overview.

According to FEMA P695 Methodology the different parameters taken into account for the development of the system archetypes were:

- ***Seismic resisting system***. The analyses were carried out for three different wall systems. The single wall solution has concentrated plasticity at the base connection. An alternative solution is coupling two different walls and distributing the dissipaters (U-shaped Flexural Plates) over the building height. A variation of the coupled walls system is the Column-Wall-Column (CWC) system where two boundary columns are used and connected to the wall element with U-shaped Flexural Plates (UFPs).
- ***Building height***. Building heights of 3, 6 and 9 storeys were taken into account, and different storey heights of 3.6m and 4.5m were assumed.
- ***Storey height***. For most archetypes a constant storey height of 3.6m of 4.5m was adopted. A variation of the 3.6m option was also considered and it is representative of an office building with commercial spaces at the ground floor.
- ***Wall depth***. Two main wall depths of 2.4m and 3.6m were assumed.

- ***Gravity loads***. The building designs were carried out considering two different floor systems. The heavy option was a Timber-Concrete Composite (TCC) floor, which comprises of timber joists supporting a concrete topping of 0.09m; a plywood panel provides the permanent formwork. The light option was a full timber Cross-Laminated Timber (CLT) floor.
- ***Seismic design category***. The archetypes in this study were designed considering mainly SDC D_{max} and D_{min}.

The Archetype buildings were divided for each resisting system into performance groups as shown in Table 1.

Table 1. System archetypes.

Group	**SDC**	**Period**	**Gravity**	***ID***	***No. storeys***	***Storey height***	***Wall depth***
PG-1	D_{max}	Short	High (TCC)	1	3	3.6	2.4
				2	3	4.5	2.4
				3	6	3.6	3.6
				4	6	5G,3.6	3.6
PG-2	D_{max}	Long	High (TCC)	5	6	4.5	3.6
				6	8	3.6	3.6
				7	8	4.5	3.6
				8	8	5G,3.6	3.6
PG-3	D_{min}	Short	High (TCC)	9	3	3.6	2.4
PG-4	D_{min}	Long	High (TCC)	10	6	3.6	2.4
				11	8	3.6	3.6
PG-5	D_{max}	Short	Low (CLT)	12	3	3.6	2.4
				13	3	4.5	2.4
				14	6	3.6	3.6
				15	6	5G,3.6	3.6
PG-6	D_{max}	Long	Low	16	6	4.5	3.6
				17	8	3.6	3.6
				18	8	4.5	3.6
				19	8	5G,3.6	3.6
PG-7	D_{min}	Short	Low (CLT)	20	3	3.6	2.4
PG-8	D_{min}	Long	Low (CLT)	21	6	3.6	2.4
				22	8	3.6	3.6

MODEL DEVELOPMENT

The multi-(axial) spring model in Figure 6 was used for the non-linear analysis. The gap opening at the wall base is simulated using a series of parallel longitudinal springs connected through rigid links. The multi-spring element was developed and introduced in Ruaumoko (Carr, 2004) for modelling post-tensioned rocking concrete systems Spieth et al. (2004), but the approach can be extended to timber systems as well.

The stiffness and strength degradations due to timber plastic deformation were simulated by assigning an elastic-perfectly plastic hysteresis with degrading gap to the multi-spring element.The post-tensioning bars were modelled using truss elements and Menegotto-Pinto hysteresis rule. The dissipation devices were modelled with non-linear springs accounting for the material stress-strain data and also low-cycle fatigue modelling. The wall elements were defined as elastic elements and rigidly linked to the P-Delta column. The latter was an elastic element infinitely rigid in the axial direction and has zero flexural stiffness.

Figure 6a shows the sketch of the model of the single wall. When modelling coupled systems the wall model was coupled to either boundary columns or another single wall for column-wall-column and coupled wall systems respectively. The wall elastic elements were rigidly connected to the dissipaters' nodes which allowed relative vertical displacement only. Sketches of the two models for the CWC and coupled walls systems are shown in Figure 6b-c.

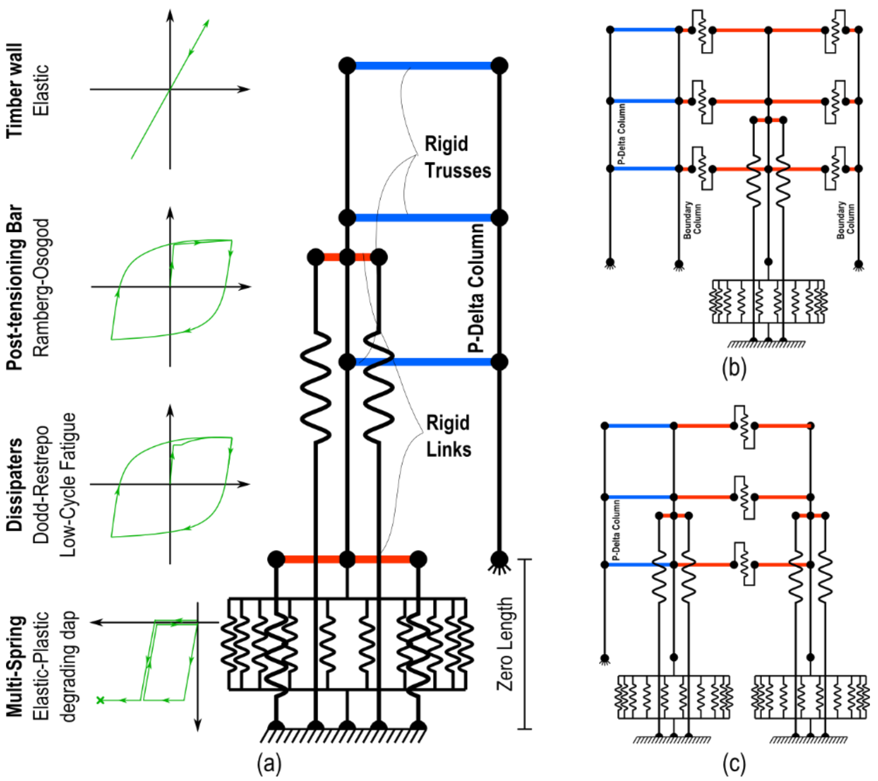

Figure 6. Multi-spring model.(a) Single wall model and material details. (b) Column-wall-column model. (c) Coupled walls model.

Figure 7 shows the comparison between experimental and numerical results in terms of moment-rotation behaviour.

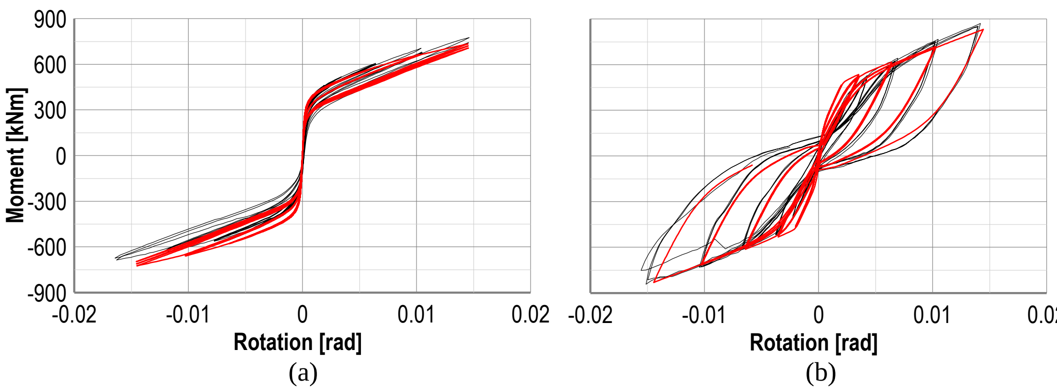

Figure 7. Numerical-experimental results comparison. (a) Single wall PT only specimen. (b) Single wall hybrid specimen. (c) Column-wall-column specimen. (d) Couple walls specimen.

NON-LINEAR STATIC ANALYSIS

Non-linear static push-over curves for each archetype provides information on the over-strength factors and the ductility of the system. In particular, the over-strength factor is defined in as:

$$\Omega = \frac{V_{max}}{V} \tag{2}$$

Where V_{max} is the maximum base shear capacity and V is the design shear capacity.
The period-based ductility of a given index archetype model is defined as:

$$\mu_T = \frac{\delta_u}{\delta_{y,eff}} \tag{3}$$

Where δ_u is the ultimate roof drift and $\delta_{y,eff}$ is the effective yield roof drift displacement. The ultimate drift/displacement is taken as that corresponding to a strength degradation of 20% or to the achivement of the maximum lateral drift capacity of the gravity system (5%).

Figure 8 reports the push-over curves for the index archetype 1 using different lateral resisting systems (respectively single wall, Column-Wall-Column and coupled walls).

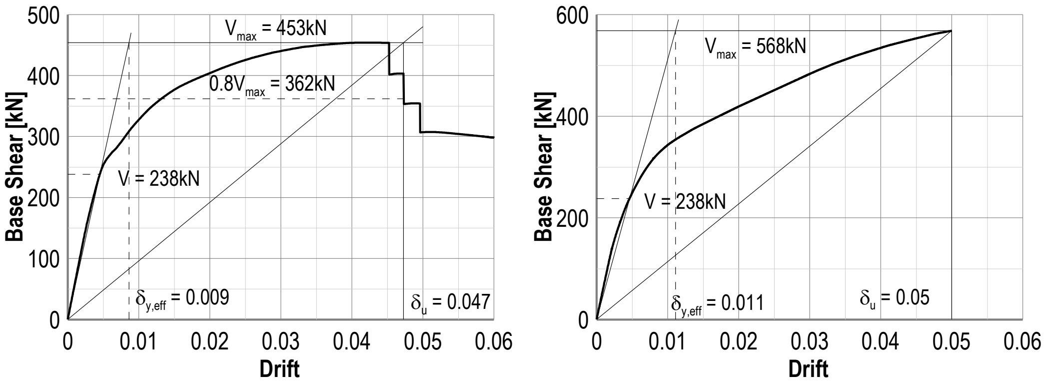

Figure 8. Static push-over analysis results (Index Archetype 1). (a) Single wall. (b) Column-Wall-Column

DYNAMIC ANALYSIS

Dynamic analyses were performed using the Incremental Dynamic Analysis (IDA), in which the individual ground motions were scaled to increasing intensities until the structure collapses (reference IDA).

Non-linear Time History analyses were carried out with the main objective of characterizing the behaviour of each archetype building in terms of Collapse Margin Ratio. The CMRwas defined as the ratio between the median collapse spectral acceleration, S_{CT}, and the MCE intensity as reported in Equation (4).

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{4}$$

The system behaviour can be significantly influenced by the spectral shape of the ground motion record set, and to account for this aspect the collapse margin ratio wass modified using a Spectral Shape Factor, SSF.

$$ACMR = SSF \cdot CMR \tag{5}$$

The Spectral Shape Factor depends on the Seismic Design Category of the structure as well as its fundamental period and period based ductility.

The parameters determining the CMR can be visualized in Figure 9a in terms of the spectral acceleration and the maximum drift. Each plot connects the result of the same ground motion record for increasing intensities.

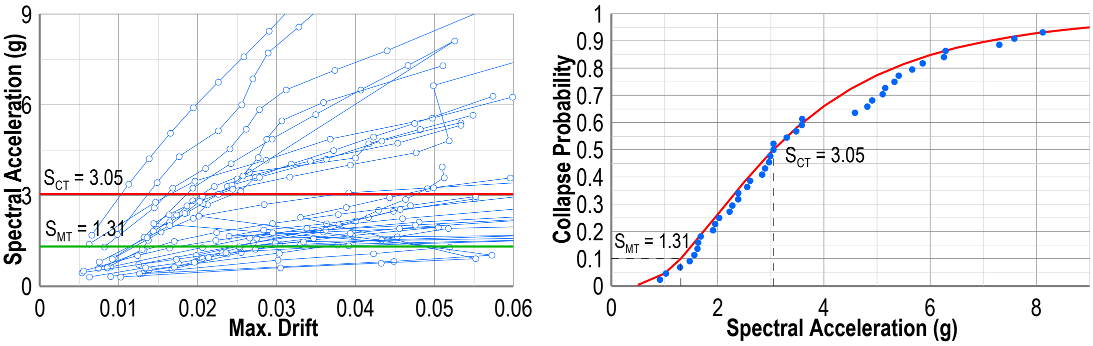

Figure 9. Incremental dynamic Analysis results (Archetype 1).

The data from IDA analysis can be used for the definition of the archetype fragility curve in Figure 9b. The curve is defined as a cumulative distribution function of the collapse spectral acceleration, and it is fitted with a lognormal function.

To assess the collapse spectral acceleration for each ground motion record, the following simulated collapse mechanisms were defined and incorporated in the model:

- The collapse of the archetype building was defined to occur when the maximum drift ratio reaches 5%. This value was assumed based on parametric analyses on the lateral drift capacity of the gravity system. The collapse of the gravity system was assessed by considering the moment and shear capacity of the gravity column when subjected to a first mode shape displacement profile. The collapse in most analyses was governed by bending failure.
- A second collapse limit was considered to be the compressive failure of the timber. A strain limit of 1% was assumed for the Laminated Veneer Lumber section according to material test data (van Beerschoten et al., 2013).
- A maximum strain limit on the post-tensioning reinforcement was also considered. A conservative value of 1% was assumed.

Earthquake Selection and Scaling

Dynamic analyses were carried out using a set of 22 earthquake records (2 components each) the record set includes strong-motion records from all large-magnitude (M > 6.5) events in the Pacific Earthquake Engineering Research Center (PEER) Next-Generation Attenuation (NGA) database (Chiou et al., 2008). Large-magnitude events dominate collapse risk and generally have longer durations of shaking. The sets included records from soft rock and stiff soil sites (predominantly Site Class C and D conditions), and from shallow crustal sources (predominantly strike-slip and thrust mechanisms).

The scaling of the records involved two steps. To remove variability between records due to inherent differences in magnitude, distance to source, source type and site conditions, the records were "normalized" to their respective peak ground velocities. The records were then scaled such as the media spectral acceleration matched the MCE spectral acceleration at the fundamental period.

Scaling factors were chosen relatively to the evaluated fundamental period, which, according to ASCE 7-10 requirements, is a function of the building height.

PERFORMANCE EVALUATION

Based on the total system uncertainty values, the acceptable values of the Adjusted Collapse Margin Ratio were taken as 10% and 20% acceptable collapse probability. The trial R-factor assumed in the archetype design phase was confirmed whether the Adjusted Collapse Margin Ratio values satisfies the following conditions:

- The collapse probability for the each performance group shall not exceed 10%. This condition is satisfied when the average ACMR value across the performance group is greater or equal to the acceptable value at 10%, $ACMR_{10\%}$.

$$\overline{ACMR} \geq ACMR_{10\%}$$

- The collapse probability for each index archetype building shall not exceed 20%; therefore, the ACMR value for each archetype shall be greater or equal to $ACMR_{20\%}$.

Table 2 reports the analysis results compared to the acceptable values for every lateral resisting system considered. The results of the non-linear static analyses are listed as well.

Table 2. Summary of the performance evaluation for the Single Wall system

		Single Wall					***CWC***					***Coupled walls***				
Group No.	***ID***	***Ω***	***$Ω_0$***	***CMR***	***ACMR***	Check	***Ω***	***$Ω_0$***	***CMR***	***ACMR***	Check	***Ω***	***$Ω_0$***	***CMR***	***ACMR***	Check
PG-1	1	2.54	2.61	1.67	2.19	pass	2.92	3.01	2.70	3.57	pass	1.50	3.13	2.16	2.77	pass
	2	2.66		1.99	2.45	pass	3.33		3.04	4.03	pass	1.50		2.48	3.19	pass
	3	2.62		2.20	2.74	pass	2.86		3.19	4.30	pass	1.31		2.53	3.34	pass
	4	2.64		2.23	2.83	pass	2.91		2.93	4.01	pass	1.25		2.82	3.76	pass
PG-2	5	2.50	2.65	2.42	3.02	pass	2.74	3.05	3.34	3.80	pass	1.11	3.19	2.70	3.67	pass
	6	2.64		2.19	2.77	pass	3.20		5.79	6.77	pass	1.06		2.89	3.95	pass
	7	2.67		2.62	3.45	pass	2.98		6.50	7.91	pass	0.90		3.19	4.46	pass
	8	2.79		2.29	2.91	pass	3.30		2.00	2.57	pass	1.02		2.98	4.10	pass
PG-3	9	2.41	2.41	2.73	2.96	pass	2.81	2.81	2.41	3.14	pass	0.69	3.58	3.64	4.09	pass
PG-4	10	2.66	2.77	5.05	4.73	pass	2.69	2.92	2.51	3.37	pass	0.41	3.61	5.95	6.90	pass
	11	2.87		4.78	5.67	pass	3.15		2.75	3.67	pass	0.33		5.86	7.00	pass
PG-5	12	2.61	2.57	1.77	2.35	pass	2.61	2.76	2.70	3.49	pass	1.50	3.50	2.16	2.77	pass
	13	2.40		1.86	2.22	pass	2.71		3.04	4.17	pass	1.50		2.47	3.18	pass
	14	2.65		2.12	2.65	pass	2.84		3.18	4.32	pass	1.31		2.71	3.58	pass
	15	2.61		2.33	2.91	pass	2.89		2.81	4.02	pass	1.25		2.82	3.76	pass
PG-6	16	2.55	2.62	2.27	2.93	pass	3.63	3.13	3.62	4.13	pass	1.11	3.16	2.84	3.86	pass
	17	2.60		2.40	3.08	pass	3.14		5.79	6.87	pass	1.06		2.76	3.78	pass
	18	2.61		2.70	3.59	pass	3.11		5.86	7.00	pass	0.90		3.27	4.57	pass
	19	2.72		2.39	3.11	pass	2.63		2.70	3.57	pass	1.02		2.98	4.10	pass
PG-7	20	2.57	2.57	3.17	3.61	pass	3.06	3.06	3.04	4.03	pass	0.69	3.37	3.86	4.33	pass
PG-8	21	2.64	2.76	4.79	5.34	pass	2.36	2.89	3.19	4.30	pass	0.41	3.41	5.79	6.71	pass
	22	2.88		5.02	6.09	pass	3.42		2.93	4.01	pass	0.33		5.86	7.00	pass

DETERMINATION OF THE SEISMIC PERFORMANCE FACTORS

The seismic performance of the system was defined in terms of response modification factor, *R*, deflection amplitude factor, C_d, and system over-strength factor, $Ω_0$.

The trial value of the response modification factor R = 7 assumed during the design phase was confirmed for all the performance groups in Table 2.. Table 2 also reports the over-strength factors resulting from quasi-static analyses.

The system over-strength factor, $Ω_0$, was taken as less than the largest average value of the calculated over-strength for each performance group, *Ω*, and rounded to half unit intervals (e.g., 1.5, 2.0, 2.5, and 3.0); therefore, the systems over-strength factor can be taken as 3.5.

The deflection amplification factor, C_d, was evaluate as:

$$C_d = \frac{R}{B_I} \tag{2}$$

Where B_I is a numerical coefficient from ASCE 7-10 depending on the critical damping of the system and the fundamental period. For the system under investigation the assumed critical damping is assumed to be 3%, so B_I = 0.93; therefore, the deflection amplification factor is equal 7.5.

CONCLUSIONS

An extensive parametric analysis was carried out according to the FEMA P695 procedure to determine the seismic performance factors of post-tensioned rocking-dissipative timber systems.

Extensive experimental work was carried out in references) (Kelly, 1972; Iqbal et al., 2007; Van Beerschoten et al., 2010; Sarti et al., 2013; Baird et al., 2014; Sarti et al., 2014) providing the necessary required information and the basis for the model calibration. Design requirements according to ASCE 7-10 were adopted, and also ad hoc system design limit states were proposed as part of this study.

Alternative archetypes were developed and designed according to the FEMA P695 methodology considering a comprehensive set of parameters. The latter included the seismic resisting system and dimensions, building height, storey height, gravity loads and seismic design category. Non-linear models were developed for each archetype building on the basis of the required information. The model included the full range of the behavioural aspects affecting the lateral resisting system; moreover, non-simulated collapse mechanisms accounting for the lateral drift capacity of the gravity framing were incorporated. Non-linear quasi static and dynamic analyses were carried providing information on the system performance in terms of over-strength factor, ductility and collapse probability.

The results from dynamic analyses confirmed the trial response modification factor assumed in the design phase; therefore, the resulting seismic performance factors were:

- Seismic Response factor R = 7.0
- Deflection amplification factor C_d = 7.5
- System over-strength factor Ω_0 = 3.5

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