Pres-Lam in the US: the seismic design of the Peavy Building at Oregon State University

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2017 NZSEE Conference

ABSTRACT: Pres-Lam is a post-tensioned rocking timber technology that has been developed over the last decade at the University of Canterbury. Pres-Lam overcomes a major challenge in timber construction, the development of a high strength moment connection, by tying mass timber elements together with high-strength steel post-tensioned tendons. In seismic areas, additional reinforcing can be added to the system increasing capacity as well as providing hysteretic damping.

In 2010 Pres-Lam moved from laboratory testing to onsite implementation and has now been used in the construction of numerous building in New Zealand and around the world. This paper will present the lateral load design of the first Pres-Lam structure to be built in the United States: the Peavy Building at Oregon State University, Corvallis, Oregon.

Peavy is a three-storey mass timber building within the College of Forestry. A glulam and CLT gravity structure support the timber-concrete-composite floor, which is made up of CLT panels spanning between glulam beams. The lateral load carrying capacity is provided in the two orthogonal directions by Pres-Lam walls fabricated from Cross Laminated Timber (CLT).

The paper will present an overview of the design philosophy and the main motivations for the use of the Pres-Lam system, discuss the requirements for U.S. code compliance, and review the nonlinear time-history analysis of the Pres-Lam structure.

1 INTRODUCTION

Timber is at its renaissance as a structural material and is globally trending towards open-plan buildings. The increasing demand for commercial timber buildings requiring flexibility within the building plan poses the main issue of creating strong connections to resist the lateral loads resulting from wind and seismic actions.

Post-tensioned timber systems (Pres-Lam systems), first developed in the 2000s (Palermo *et al.*, 2005) and extended from the PRESSS systems (Priestley, 1996), represent a suitable and convenient way of connecting structural elements to create high-capacity and low-damage moment-resisting connections.

When a reduction of seismic design forces is required, the system provides a combination of re-centering and energy dissipation, which allows the minimization of post-event residual displacements while providing hysteretic damping to the system.

Extensive experimental and numerical research was performed on a comprehensive set of structural elements, both walls or frames, as well as on scaled buildings (Palermo *et al.*, 2006; Newcombe *et al.*, 2010; Smith *et al.*, 2013; Smith *et al.*, 2014; Sarti *et al.*, 2015). Experimental results proved the excellent performance of the system, which was able to localize the damage to sacrificial elements, while leaving the structural elements in an elastic and undamaged state.

Figure 1. Nelson and Marlborough Institute of Technology.

Following extensive research on this low-damage structural system, several buildings have been constructed in New Zealand using the Pres-Lam system.

The Nelson and Marlborough Institute of Technology (Nelson, New Zealand) is a three-storey timber building whose lateral-load resisting system consists of Pres-Lam post-tensioned coupled walls (Devereux *et al.*, 2011).

Another more recent example is the Trimble Navigation Building, which was the first Pres-Lam building of the Christchurch Rebuild (Brown *et al.*, 2012). The building is a two-storey structure with post-tensioned timber frames and coupled walls in the two principal directions.

Figure 2. Trimble Navigation Office.

Starting from the first New Zealand built examples, the system has spread worldwide with buildings constructed in Switzerland, Japan and soon in the United States.

This paper presents the seismic design of the Peavy building at Oregon State University, the first Pres-Lam building to be constructed in the United States.

In the first part, the paper provides an overview of the gravity and lateral load resisting systems and the main motivations of the use of the Pres-Lam technology.

The final part of the paper presents a detailed overview of the three-dimensional nonlinear time-history analysis procedure carried out to prove code compliance.

2 STRUCTURAL FORM

2.1 Gravity load resisting system

As a College of Forestry building, one of the mandates for the structure and architecture is to celebrate and showcase the wood. The structural system was developed out of consideration for cost, use of local materials and products, and integration of services. The building is 3 stories tall with entirely timber and timber-concrete-composite construction above the ground floor. Products used within the Peavy building include glulam columns and beams, CLT panels and walls, plywood, and LVL sheathing. Connectors are primarily custom steel concealed hangers and self-tapping screws. The basic floor structure is a timber-concrete-composite slab with concrete topping on CLT, connected together using HBV steel mesh developed by Ticomtec GmbH (Ticomtec, 2017). The slab is supported by double glulam beams on flat with gaps between beams for service integration (Figure 3). The floor system is designed to maximize exposed wood and integrate services. Gaps between the glulam beams and the

CLT panels are used throughout the building to conceal sprinklers, wiring, and ducts.

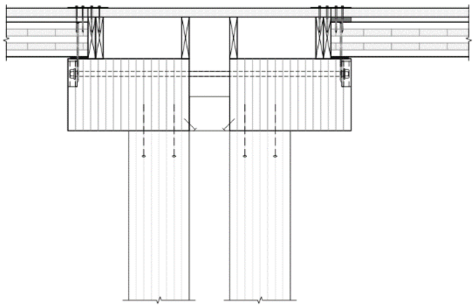

Figure 3. Double glulam beam.

2.2 **Lateral load resisting system**

Peavy building is intended to be a demonstration project for wood innovation, as well as a learning and teaching tool for students attending the University. The innovation of a post-tensioned shear wall structural system appealed to the client and stakeholders involved in the building, as did the building's capacity for self-centring after an earthquake. In addition to meeting the code requirements for a lateral system, the Pres-Lam system is designed with a capacity for reparability after a design-level earthquake.

With the requirement of large open-plan spaces and a limited number of lateral load resisting elements, the increased connection capacity Pres-Lam system also allowed a significant reduction of walls (i.e. approximately a 50% reduction compared to traditional a CLT wall system).

2.2.1 *Shearwalls*

The shear walls are Douglas Fir 5-ply or 7-ply CLT walls. CLT panels were the material of choice because the first American CLT production facility has recently opened in Riddle, Oregon, 200km away from the building location. The building reaches 13.5m tall which is slightly taller than the longest panel that can be made, so the shear walls contain a horizontal splice above the third floor. The walls are 2.75m wide to accommodate the 3m grid spacing and no vertical splices are required between walls. Walls are either single walls or double walls with energy dissipaters (see Section 2.2.3) between them.

2.2.2 *Post-Tensioning*

All walls contain four steel Dywidag 150 grade post-tensioned bars with a post-tension load of up to 225kN / bar. These bars, once tensioned, create adequate moment capacity of the shear-wall to resist a major seismic event and return the wall to centre. The arrangement of bars depends on the thickness of the CLT panel. The bars are anchored into the concrete grade beam and are spliced about 750 mm up from the grade beam. The bars are anchored to the top of the panel with a steel bearing plate (Figure 4). At the top of panel, at the anchor location, the panel is spliced horizontally and fastened with HSK steel plate connectors by Ticomtec GmbH (Ticomtec, 2017).

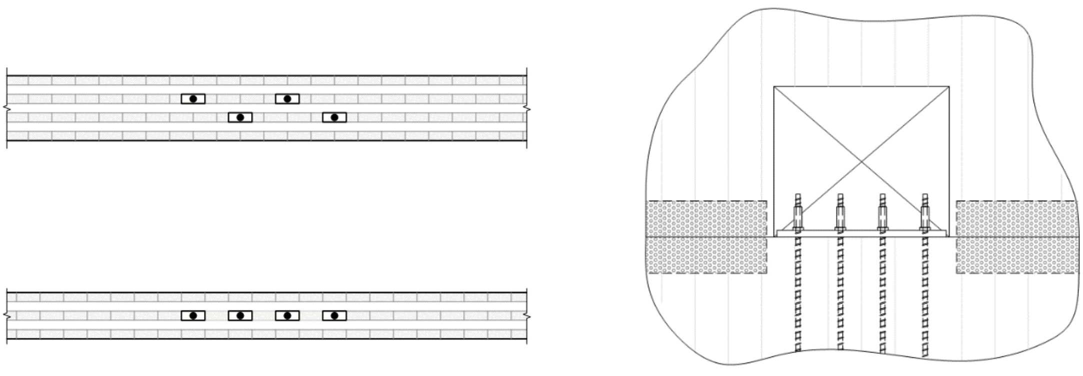

Figure 4. Post-tension rods in CLT wall (left) and rod connection and CLT splice (right)

2.2.3 *Energy Dissipation*

Energy dissipation devices are included in the form of U-shaped Flexural Plates (UFPs) (Skinner *et al.*, 1974) that are used either as hold-downs, connectors between walls and columns, or connectors between walls (Figure 5). These steel yielding elements are connected to the shearwalls using steel HSK plates glued into slots in the walls.

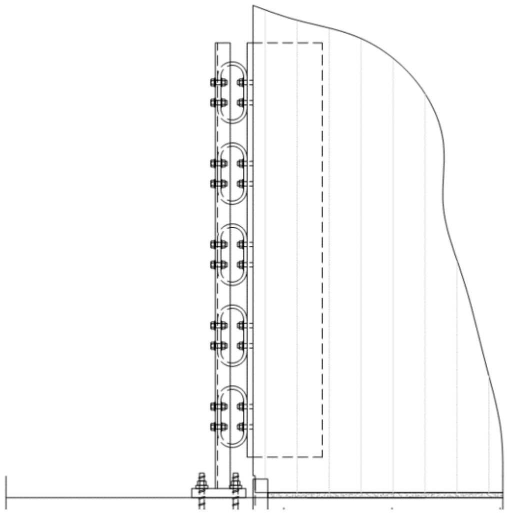

Figure 5. UFP hold down / energy dissipater

2.2.4 *Diaphragm*

The diaphragm at the roof level consists of LVL sheathing. LVL was a compromise between a heavier and more expensive 3-ply CLT diaphragm, and an inexpensive plywood diaphragm that was not sufficient for the high diaphragm design forces. The roof is divided into two levels: a low roof academic bar, and a high roof atrium.

The low roof academic bar contains 1.2m wide CLT panels spaced at 1.8m, so the diaphragm consists of 44mm thick LVL on the CLT, connected together using screws, and is considered to be rigid. Wood strapping is used between the CLT and sheathing to avoid trapping moisture between the two wood elements. The sheathing is connected to the shearwalls using screws. LVL beams on flat are used as chords and collectors (Figure 6 left).

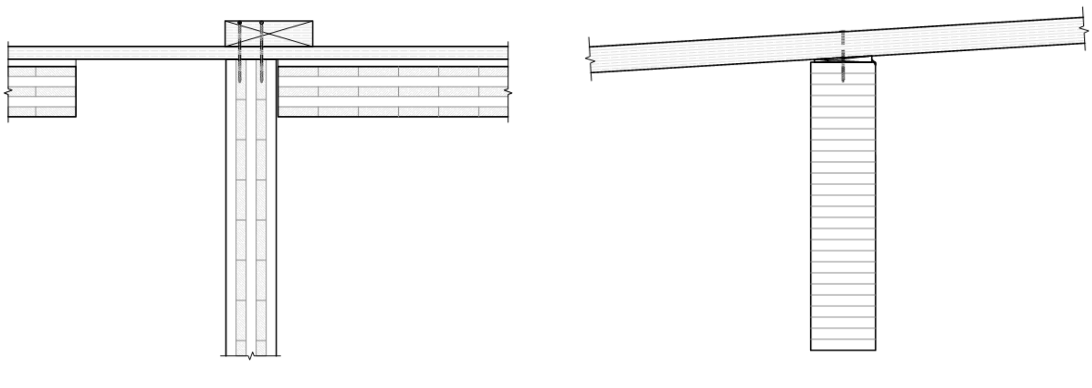

Figure 6. Academic bar roof diaphragm to shearwall connection (left). Atrium roof sheathing at glulam beam (right)

The high roof atrium structure does not contain CLT, and instead has glulam beams at 3m spacing with 89mm thick LVL sheathing. This thicker sheathing is connected together using steel plates that are sawcut into the panels to interlock them together. The atrium diaphragm was considered to be rigid in the numerical analysis (Figure 6 right).

The level 2 and level 3 diaphragms consist of the 89mm thick concrete topping used as part of the timber-concrete-composite floor system. The concrete topping is reinforced with rebar to provide the tensile and shear strength required. A steel T section connects the diaphragm to shearwalls for in-plane shear forces. The concrete topping and steel diaphragm connectors are jointed to allow the shearwalls to lift without failure of the concrete dipahagm.

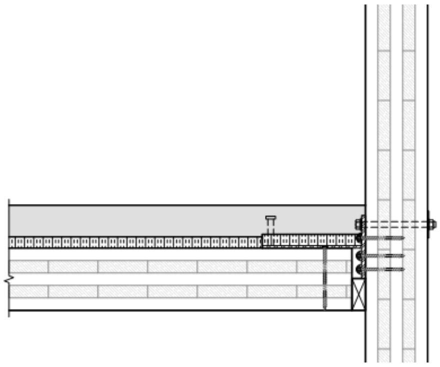

Figure 7. Concrete diaphragm connection to shearwall

3 LATERAL LOAD DESIGN

As Pres-Lam is not currently include in the American standards, to prove code compliance in accordance to the provisions of ASCE 7-10, a nonlinear three-dimensional model was used. In particular, due to the lack of system-specific response modification factor, deflection amplification factor and over-strength factor, the numerical results were considered for strength and displacement verifications of the structure.

In accordance to Chapter 16 of ASCE 7-10 (2010) (i.e. seismic response history procedure), the average response across the set of earthquake records was considered for the design. Where the consideration of seismic load effects including overstrength factor was to be considered the maximum response values obtained from the suite of analyses was used.

The following sections briefly summarize the nonlinear time history analysis procedure used for the design of the Peavy building.

3.1 Numerical model

To simulate the nonlinear response of the Pres-Lam walls, a lumped plasticity modelling approach was used, which combines the use of elastic structural elements with springs which represent the non-linear (gap opening) rotations in the system.

Single Pres-lam walls were modelled through a pair of parallel rotational springs simulating the combination of re-centering (multi-linear hysteresis) and dissipation (elastic plastic hysteresis) at the wall base (Figure 8a).

The lumped plasticity model for the coupled walls included the re-centering behaviour at the wall base, while the yielding elements were simulated by distributed elastic-plastic springs activated by the relative vertical displacement between the two wall elements (Figure 8b).

The modelling approach has been extensively validated against experimental results by Sarti et al. (2015) and Iqbal et al. (2007) for both single and coupled walls and is capable of accurately predicting the non-linear cyclic behaviour of the system as shown in (Figure 8c).

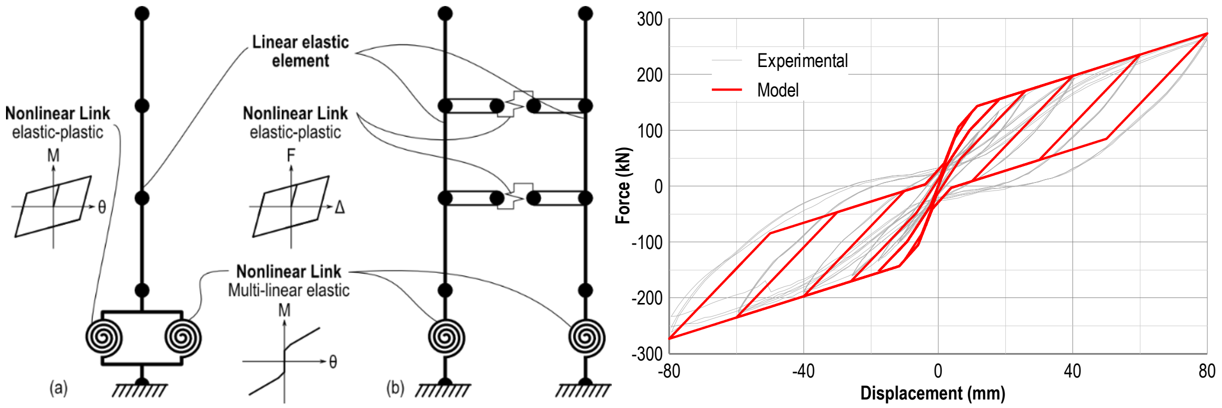

Figure 8. Shear walls nonlinear models. (a) Type A and (b) Type B. (c) Comparison of the model against experimental results by Sarti et al. (2015).

The material properties used to calibrate the lumped plasticity models are reported in Table 1.

Table 1. Material properties.

Cross-Laminated Timber		**Post-tensioning steel**	
Description	Notation and value	Description	Notation and value
Modulus of elasticity	E = 10.3GPa	Modulus of elasticity	E = 205.9GPa
Shear modulus	G = 690MPa	Minimum tensile stress	f_{pu} = 1,035MPa
Bending strength	f_b = 28.9MPa	**U-shaped Flexural Plates (UFPs)**	
Tensile strength (parallel)	f_t = 19MPa	Modulus of elasticity	E = 200MPa
Compr. strength (parallel)	f_c = 28.2MPa	Tensile strength	f_t = 400MPa
Shear strength	f_v = 3.6MPa	Yield point	f_{ty} = 248MPa

The numerical models of the single and coupled walls were used to create a three-dimensional model (see Figure 9) used for the non-linear time-history analyses. As shown in Figure 9, the three-storey structure was modelled with lumped masses at each level, thus considering a rigid diaphragm assumption in accordance with Section 16.2.2 of ASCE 7-10 (2010).

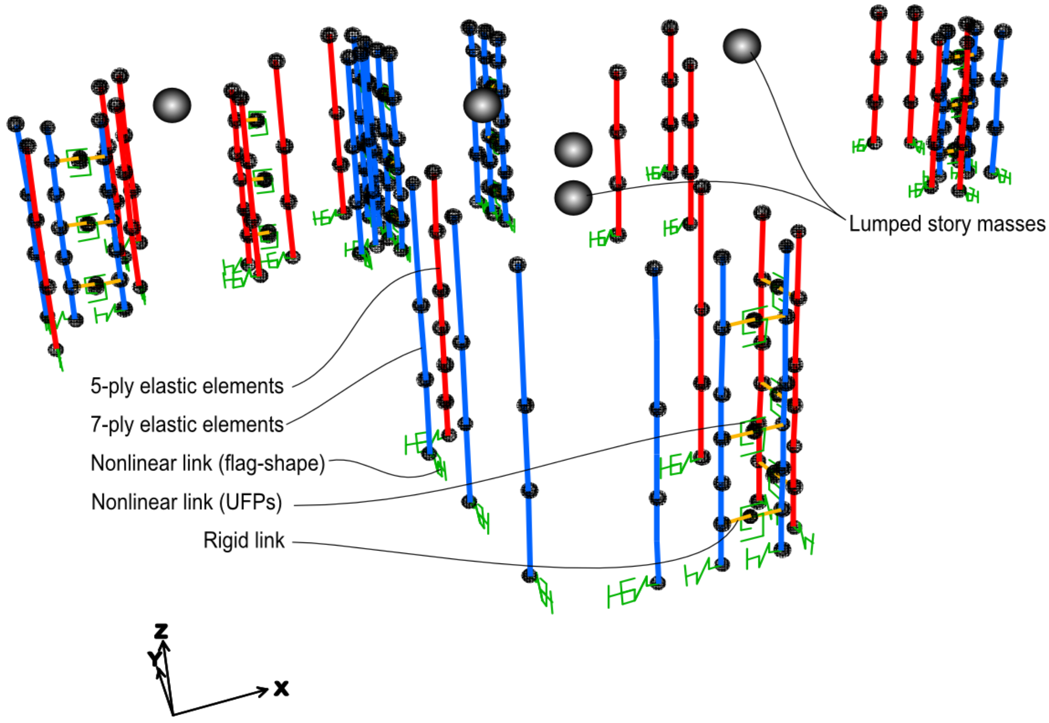

Figure 9. Model for the nonlinear response history procedure.

3.2 **Dynamic identification and seismic input**

Firstly, a modal analysis of the building was performed in order to carry out the dynamic identification of the system. The modal properties of the building are summarized below.

Table 2. Modal properties.

Mode	Period (s)	Participating mass ratio (%) EW	NS	Cumulative mass ratio (%) EW	NS	Mode type
1	1.04	75.0	0.3	75.0	0.3	1st Trans., EW direction
2	0.97	0.3	75.6	75.4	75.9	1st Trans., NS direction
3	0.53	0.8	0.1	76.1	76.0	Torsional
4	0.37	3.3	1.1	79.5	77.0	Torsional
5	0.33	1.9	4.0	81.3	81.1	Torsional
6	0.30	6.1	0.1	87.5	81.1	Torsional
7	0.26	9.3	0.5	96.8	81.7	2nd Trans., EW direction
8	0.26	0.4	15.5	97.2	97.1	2nd Trans., NS direction
9	0.16	2.8	0.0	100.0	97.1	3rd Trans., EW direction
10	0.15	0.0	2.9	100.0	100.0	3rd Trans., NS direction

A set of 7 earthquake records was selected to have characteristics consistent with those that control the maximum considered earthquake (Cascadia subduction rupture) and scaled in accordance to Chapter 16 of ASCE 7-10 (2010). A summary of the selected record is shown in Table 3 reporting the event name, year, station name, magnitude and scaling factor.

The scaling factors were evaluated to minimize in a least mean square sense the difference between the SRSS spectrum of each record and the design spectrum in the period range of 0.2T and 1.5T, where T is the fundamental period of the structure. The evaluation of the scaling factors also accounted for the requirement of Section 16.1.3.1 according to which the average SRSS spectrum from all the records shall not fall below the corresponding ordinate of the design response spectrum in that range.

As a three-dimensional analysis was considered, the model was subject to bi-directional loading in accordance to Chapter 16 of ASCE 7-10. The model was subjected to both components of the ground motions in the two directions of response (i.e. (a) component 1 in the EW direction and component 2 in the NS direction, and (b) component 2 in the EW direction and component 1 in the NS direction).

Table 3. Record set of Nonlinear Response History procedure.

ID	Earthquake Name	Year	Station Name	PGA	M_w	Scaling Factor	Type
1	Sth. Peru	2001	Moquegua	0.38	8.4	1.28	SZ
2	Chile	2010	Hualane	0.59	8.8	0.73	SZ
3	Tohoku	2011	Chohshi	0.56	9.0	1.63	SZ
4	Tohoku	2011	Ishige	0.35	9.0	0.92	SZ
5	Tohoku	2011	Tajiri	0.37	9.0	1.23	SZ
6	Landers	1992	North Palm Springs	0.45	7.3	2.33	C
7	Kocaeli	1999	Zeytinburnu	0.41	7.5	2.60	C

NOTE: SZ = subduction zone record, C = crustal record

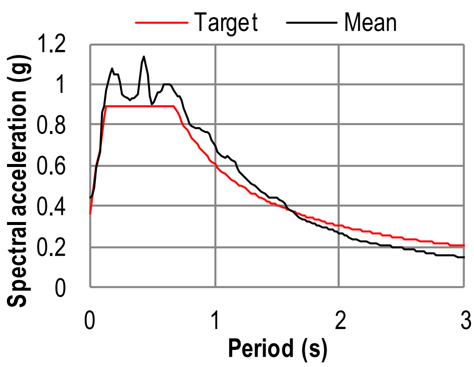

Figure 10. Average SRSS spectrum of the record set for Nonlinear Response History Procedure.

3.3 **Results**

A selection of results from the nonlinear time-history analysis is shown in Figure 11.

Figure 11a shows the force-displacement plot resulting from one of the nonlinear time history analysis (record 2) and compares the dynamic response to the static backbone curve. As the chart shows, the dynamic force-displacement plot fluctuates around the backbone static response highlighting the influence of higher modes on the structural behaviour. The force-displacement loop also shows that nonlinear behaviour is occurring which is highlighted by the flag-shaped hysteresis.

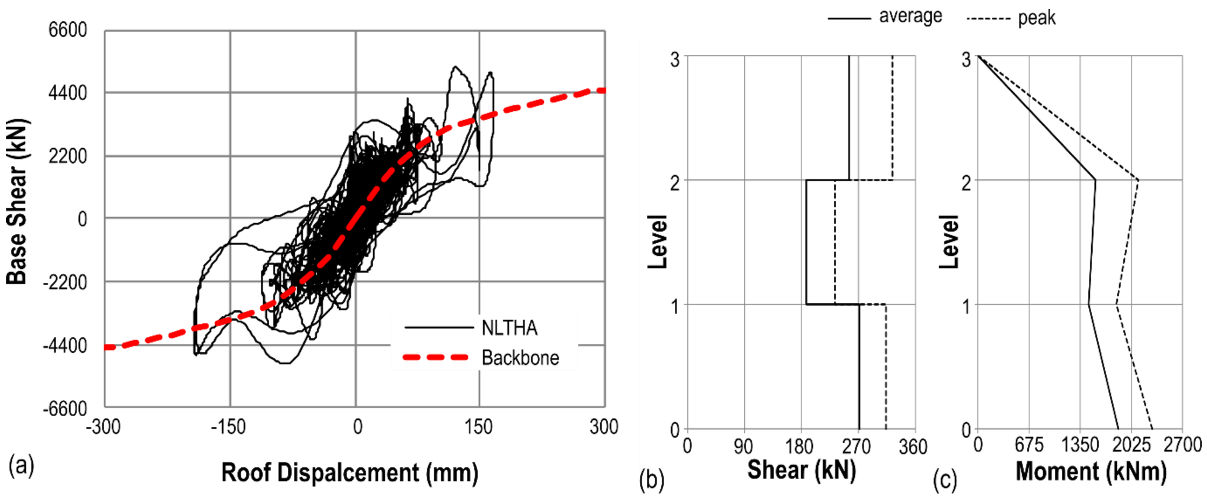

Figure 11. Analysis Results. (a) Force-displacement plot for record 2. Example of (b) Shear force and (c) bending moment envelopes.

From an element internal forces perspective, the higher mode amplification can be shown by the shear force and bending moment envelopes. The higher mode amplification on the shear force can be noted in Figure 11b, where the top level shear demand approaches the same value as the base shear. A similar behaviour can be observed for the bending moment envelope in Figure 11c. While this amplification of internal actions did not influence the design of the structural elements, it governed the design of structural details such as the diaphragm connections and wall splices.

4 CONCLUSIONS

In recent years, timber has become increasingly popular as a construction material for multi-storey open-plan buildings. The demand for flexibility within the building plan creates the need to create strong connections to resist lateral loads.

Pres-Lam systems provide a resilient and efficient way of connecting structural elements and creating high-strength joints. The system was first developed in the 2000s (Palermo *et al.*, 2005) and extended from the PRESSS systems (Priestley, 1996), and provides a combination of re-centering and energy dissipation. This allows the post-event residual deformations to be limited and to reduce of lateral load demand through hysteretic damping.

The paper presented the design of the Peavy building at Oregon State University. As a College of Forestry building, the structure was intended to celebrate and showcase the timber. The gravity load resisting system of the structure consists of a timber-concrete composite structure using Cross-Laminated Timber (CLT) panels supported by a glulam frame. Pres-Lam walls provide the lateral load resistance in the two directions. The system allows for reparability after a design-level earthquake and also permitted a significant reduction in the number of lateral load resisting elements.

The lateral load design of the structure was performed through seismic response history procedure in accordance to ASCE 7-10. Numerical results have shown that the system fully re-centered after the earthquake records and provided hysteretic damping to the system allowing the reduction of the building demand.

Higher mode effects had a significant influence on the lateral load response of the building, and resulted in the amplification of shear force and bending moment in the walls panels. This mostly affected the design of the structural details (i.e. diaphragm connections).

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