Chapitre 2

Benefits of Multiple Rocking Segments for CLT and LVL Pres-Lam Wall Systems

2.1 Résumé

Cette section présente le premier article soumis au "Journal of Earthquake of engineering" rédigé en collaboration avec Alessandro Palermo et Francesco Sarti de l'Université de Canterbury en Nouvelle-Zélande ainsi qu'Alexander Salenikovich de l'Université Laval. Cet article présente le développement des systèmes Pres-Lam à sections basculantes multiples dans le but de réduire l'amplification dynamique des efforts sismiques présente dans les systèmes à section basculante simple. De plus, une comparaison entre les considérations de conception pour des panneaux de LVL et de CLT est effectuée. Finalement, on y présente les détails de conception d'une connexion utilisée pour lier les sections des panneaux des systèmes à sections basculantes multiples.

2.2 Abstract

Press-Lam wall systems built with single rocking segments result in a dynamic amplification of forces in the structure. The objectives of this research were to compare design requirements for Pres-Lam systems made of LVL and CLT and to demonstrate that the dynamic amplification can be reduced by introducing gap openings at construction joints, thus creating multiple rocking segments. The CLT walls needed 20% wider panels and 10% to 30% less initial pre-stress in the post-tensioning bars to obtain the same moment-rotation behaviour as the LVL walls. Time-history analyses showed that the shear and bending moment envelopes were reduced

nearly 45%.

Keywords : Pres-Lam, massive timber panels, seismic design, single rocking segment, multiple rocking segments, Cross-laminated timber, Laminated veneer lumber

2.3 Introduction

In the past few years, the design philosophy in earthquake engineering took a trend to change from a concept where buildings were designed to ensure life safety only that rendered them unusable and unrepairable after a major event to a low-damage design approach [Priestley et al., 1999 ; Naeim & Kelly, 1999 ; Palermo et al., 2004]. While assuring the life safety during earthquakes, this new concept is focused on creating systems that would minimize the damage of the main structure of the building, allowing it to be usable or easily repairable after the major event. The motivation to design with this new approach started to grow after strong earthquakes that inflicted significant damage to cities around the world, like for Christchurch in New Zealand in 2011 [Buchanan et al., 2011 ; Canterbury Earthquakes Royal Commission, 2013]. The extent of the damage of the Canterbury earthquake sequence led to demolishing and rebuilding several commercial buildings, which is characterized by a loss of $30 billion including the costs of business disruption, inflation, insurance administration and rebuilding structures with higher seismic performance [Canterbury Earthquakes Royal Commission, 2013]. As the demand for tall timber buildings has been increasing due to their attractive architectural look, environmental benefits, quick assembly and high resistance to lateral loads, extensive research is necessary on design of timber buildings using the low-damage philosophy.

Within the PREcast Seismic Structural System (PRESSS) programme led by the University of California for concrete structures [Priestley, 1991 ; Priestley, 1996 ; Nakaki, 1999] a concept of a re-centering structure using a reparable plastic hinge has been advanced. To implement it, Priestley [1996], designed a hybrid connection using pre-stressed steel cables inside precast walls, beams and columns that are designed to remain elastic during seismic events, thus forcing the building to return to its original position after the earthquake. Ductile elements are positioned along the structure to provide energy dissipation for the building during major events when a gap begins to open at the ends of the rocking elements. This approach provides a combination of re-centering and energy dissipation through the rocking motion.

The PRESSS system has been adapted at the University of Canterbury [Palermo et al., 2005] to timber buildings and is known as the Pre-Stressed Laminated, or Pres-Lam, system [Buchanan et al., 2008]. In this system, pre-stressed steel bars designed to remain elastic are positioned inside of mass timber panels, such as Laminated Veneer Lumber (LVL) or Cross-Laminated Timber (CLT) or in a large glulam element, as shown on Figure 2.1. Replaceable steel dissipaters of any kind [Sarti et al., 2017 ; Smith & Pampanin, 2014 ; Skinner et al., 1974] are installed at the bottom corners of the walls and are designed to yield during the major seismic event, thus providing energy dissipation for the system. When subject to reversed cycles, this system reveals the typical flag-shaped hysteresis, schematically shown on Figure 2.1, through the combination of the elastic behavior of the post-tensioning bars and elastic-plastic behavior of the dissipaters. Different levels of re-centering can be achieved by selecting a re-centering ratio (β), which governs the hysteresis shape and can be described as the ratio of the post-tensioning moment contribution (M_{pt}) to the total moment distribution (M_tot) in the system ($\beta = M_{pt}/M_{tot}$). A minimum value of $\beta = 0.55$ is recommended by NZS 1170.5 [NZS, 2004] to provide sufficient energy dissipation and to limit the residual displacement in the structure.

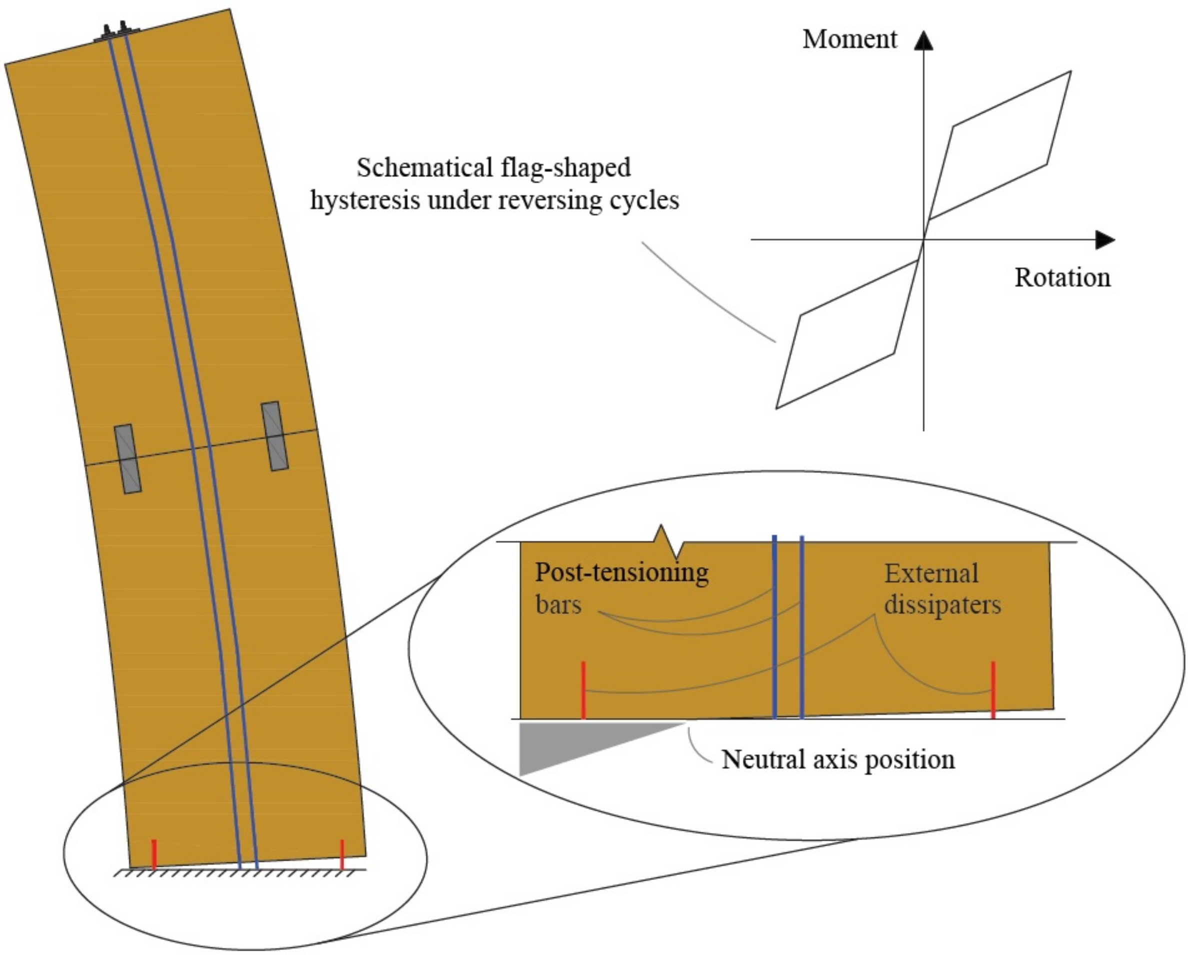


Figure 2.1 – Pres-Lam system representation

Several buildings have been built in New Zealand using the Pres-Lam technology providing low-damage lateral-force-resisting system to the structures. Figure 2.2(a) shows the Nelson and Marlborough Institute of Technology in Nelson (41.2746° S, 173.2891° E), which is the first building constructed with this technology using LVL panels [Devereux et al., 2011]. This three-storey building was designed using coupled walls with U-shaped flexural plates (UFPs) [Skinner et al., 1974], which provide the energy dissipative source for the system. Figure 2.2(b) shows the Kaikoura District Council Building that was built in the city of Kaikoura (42.40538° S, 173.68389° E) in 2014 using the single rocking wall system. This three-storey building is the first to use this technology with the CLT as a core combined with the LVL at the edges. Other examples not herein reported for the sake of brevity are the Trimble Building and Carterton Event Centre (41.0259° S, 175.5287° E) [Curtain et al., 2012].

FIGURE 2.2 – Buildings and systems examples. (a) Nelson and Marlborough Institute of Technology. Coupled walls (adapted from Sarti, 2015) (b) Kaikoura District Council Building. Single walls (adapted from Sarti, 2015)

The original concept developed at the University of Canterbury used single rocking wall segments made of LVL panels, which possess considerable compressive strength ($f_c = 45MPa$) and rigidity ($E = 11000MPa$). Due to the popularity of CLT across the world and based on the research by Dunbar et al., 2014, further studies of the Pres-Lam technology using this type of panel is needed. The CLT panels consist of multiple orthogonal layers of sawn lumber bonded with structural adhesives. The standard CLT panels are not as strong and not as rigid as LVL because of the transverse layers loaded perpendicular to grain, which needs to be considered in the design. CLT was used in combination with LVL for the Pres-Lam walls of the Kaikoura District Council Building. Due to a lower compressive strengh of CLT laminaitons parallel to grain ($f_c = 19.3MPa$), LVL was used at the edges of the walls to limit the crushing of timber during rocking motion.

The first buildings with the Pres-Lam system have been limited to three storeys in height with the wall panels spanning the entire height of the building. Therefore, there was no need for construction joints between the wall panels of the lower and the upper storeys. However, one of the concerns in the design of taller timber buildings is their higher flexibility than concrete structures, which makes them more susceptible to a dynamic amplification of the forces in the upper storeys. Original wall designs were performed to allow the formation of a plastic hinge at the base of the structure assuming that the forces in the upper structure would not increase once the hinge was formed. However, further research has shown that the forces can notably increase after the formation of the plastic hinge, because of the effects of higher modes [Priestley & Amaris, 2002]. The traditional solution would be to design the shear wall as a single rocking segment with the analogy of a continuous vertical cantilever with a ductile joint at the base of the rocking wall element of a large cross section resisting the amplified forces along the height of the structure. In taller buildings, this approach would require perfectly rigid connections at construction joints between the wall panels stacked on top of each other. This concept leads to costly connections using numerous fasteners that also slow the overall building construction. An alternative solution, known as the multiple rocking segment system, has been proposed for concrete walls [Wiebe & Christopoulos 2009 ; Qureshi & Warnitchai 2016] aiming at reducing the dynamic amplification in the upper storeys by introducing connections at construction joints that allow a gap opening, thus simplifying the connection and reducing the costs.

The objectives of the present paper are as follows. First, to compare the design requirements for the systems with single rocking segments made of LVL and of CLT panels aiming to obtain the same moment-rotation behavior for both systems through analytical modeling of several multi-storey case study buildings. Second, to assess the seismic forces induced in the structures of both types and to quantify the effects of higher modes on single rocking segment systems. Third, to demonstrate that these effects can be reduced by allowing gap openings between the panels using simple connections at the construction joints, thus creating multiple rocking segment systems. Numerical models have been developed for the single and multiple rocking segment systems and non-linear time history analyses (NLTHA) have been performed using OpenSEES [MecKenna, 2011].

2.4 Methodology

2.4.1 Case Study Buildings

To assess the behavior of the systems with single and multiple rocking segments made of LVL or CLT, eight case study building configurations have been considered representing office buildings with variable number of storeys (3, 6, 8 and 9) and inter-storey height (3.6-m and 4.5-m) [Sarti et al., 2015]. One of the buildings has been designed with a 5-m high ground floor considering the possibility of accommodating a commercial space. The structures consisted of three 6-m bay frames, for a total of 18 m in the transverse direction, and of four 8-m bay frames, for a total of 32 m in the longitudinal direction (see Figure 2.3). The lateral-force-resisting systems consisted of several post-tensioned walls with concentrated plasticity at the base provided by external steel dissipaters at the ends (see Figure 2.3). The number and position of the walls were the same for both LVL and CLT systems. The gravity resisting systems consisted of glulam timber posts and beams with timber-concrete composite (TCC) floors.

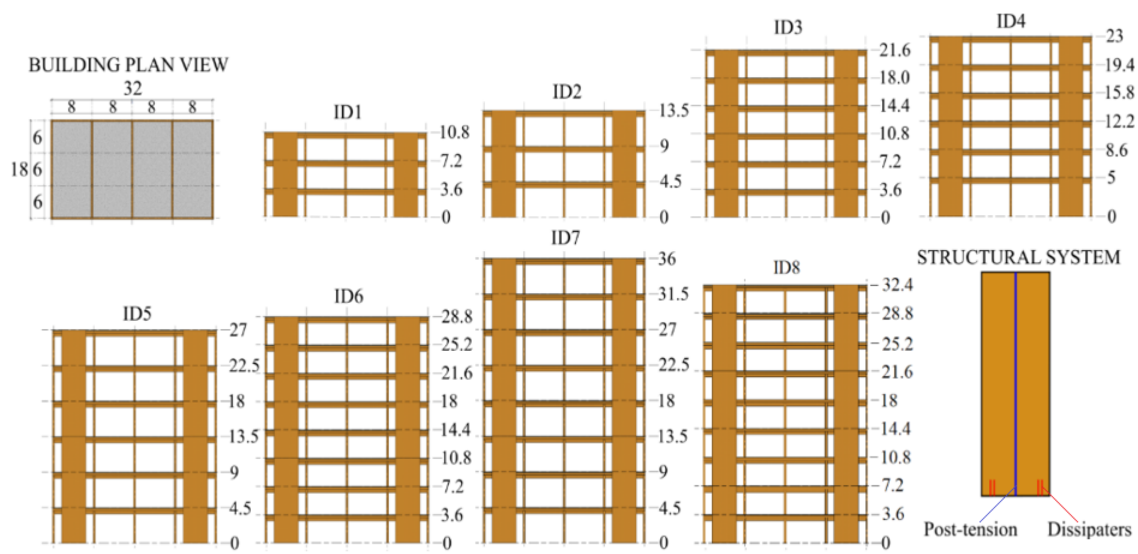


Figure 2.3 – Case study buildings

2.4.2 Seismic Design Loads

To determine the design loads on the walls, a Displacement-Based Design (DBD) procedure [Priestley et al., 2007] in accordance with NZS 1170.5 [NZS, 2004] has been performed for each case study building as in Sarti [2015]. The buildings have been designed with a ductility ratio $\mu = 2.5$, defined as the ratio of the ultimate and the yield displacements at the roof level, and with an hysteretic damping (ξ_{hyst}) determined as follows :

$$\xi_{hyst} = k_\xi \frac{(2-2\beta)(\mu-1)}{\mu\pi(1+r(\mu-1))}$$

where k_ξ is the area-based hysteretic damping modification factor and r is the post-yield stiffness factor. The location used for the analyses is the city of Christchurch in New Zealand (Z=0.3) with a soil type D. To obtain the displacement design spectrum (S_d) an importance level of IL2, a return period (R) of 1.0 and a structural performance factor (S_p), of 1.0 have been taken. The structure is then characterized by a secant period (T_e), a secant stiffness (K_e) and an equivalent height (H_e). Using the design drift (θ_d) of 1.2% and all the properties above, the design base shear (V_b) and design base moment (M_b) have been determined to design the walls of the buildings. The design parameters and results are shown in Table 2.1.

TABLE 2.1 – Design seismic loads

ID	No. Of Storeys	h_1 [1] (m)	h_i [2] (m)	H [3] (m)	Storey Mass (Ton)	V_b (kN)	H_e (m)	M_b (kNm)
1	3	3.6	3.6	10.8	320	3946	8.4	33146
2	3	4.5	4.5	13.5	320	3475	10.5	36488
3	6	3.6	3.6	21.6	320	5128	15.6	79997
4	6	5	3.6	23	320	4979	16.7	83149
5	6	4.5	4.5	27	320	4108	19.5	80106
6	8	3.6	3.6	28.8	320	5148	20.4	105019
7	8	4.5	4.5	36	320	4118	25.5	105009
8	9	3.6	3.6	32.4	320	5682	22.9	130400

1 : Height of the first storey
2 : Inter-storey height
3 : Total height of the building

2.4.3 NLTHA Parameters

Once the design of the walls has been completed, they were subjected to NLTHA for further investigations of the influence of higher mode effects on the performance of the walls. A total of ten ground motion records have been selected from the Pacific Earthquake Engineering Research Center (PEER) Next-Generation Attenuation (NGA) database [Chiou et al., 2008]. The records chosen are strong-motion earthquakes with a magnitude (M) higher than 6.5 to match a highly probable event in Christchurch. Ground motions are then scaled for each case study building for both LVL and CLT systems, based on NZS 1170.5 [NZS, 2004] methodology. With this method, it is required to scale the records spectra obtained from each ground motion to match the design spectrum selected between $0.4T$ and $1.3T$, where T is the fundamental period of the system. The fundamental period of each wall is obtained by performing modal analysis on the multiple-degree-of-freedom numerical models. Figure 2.4 shows a list of the chosen strong-motion records and an example of the mean scaled records spectrum compared to the design spectrum for a wall of case study building 5 in CLT with a fundamental period of T=1.18s.

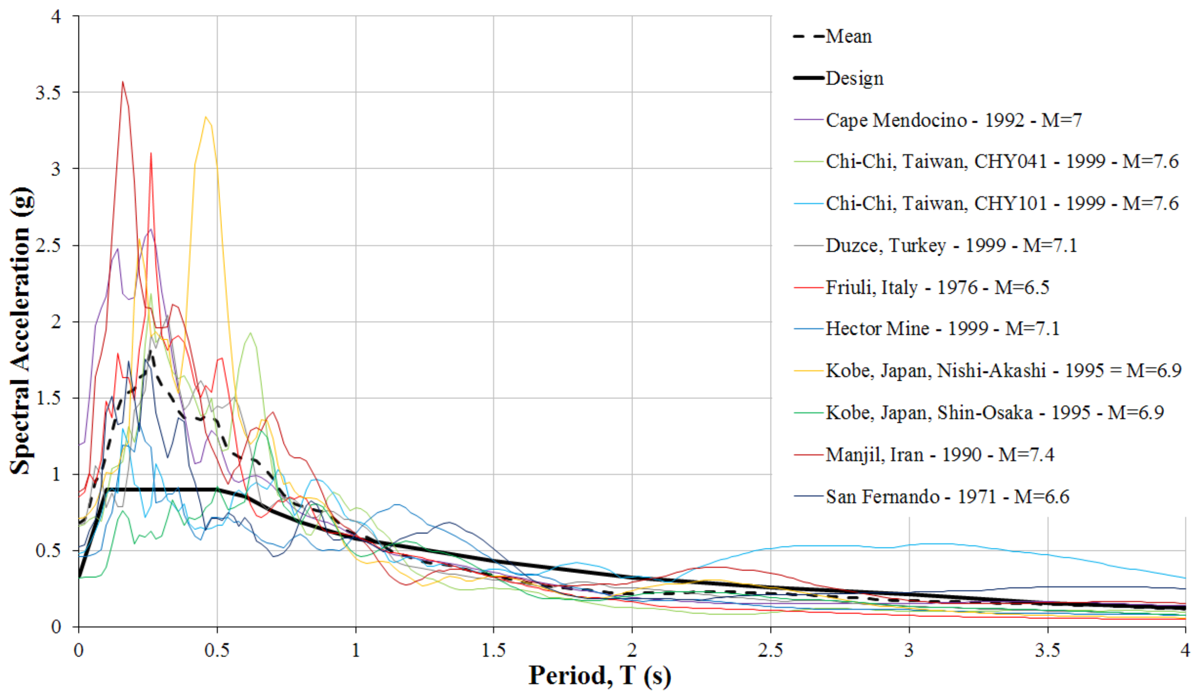


FIGURE 2.4 – Mean scaled records spectra compared to design spectrum for case study building 5 (CLT) (scaled at $T = 1.18s$)

2.4.4 Numerical Modeling

To obtain the seismic forces induced in the structures and to demonstrate the effects of higher modes on single rocking segment systems using the NLTHA, multi-spring numerical models have been developed in OpenSEES [MecKenna, 2011] for both types of wall panels and calibrated to match the analytical study presented in the previous section. The multi-spring model was first developed by [Spieth et al., 2004] for precast pre-stressed concrete frame structures and can be applicable to rocking systems in general including the post-tensioned timber wall rocking system [Sarti, 2015 ; Sarti et al., 2017 ; Iqbal et al., 2007]. The contact between the base of the wall and the foundation is modeled with several parallel zero-length springs allowing a gap opening to be considered in the analysis (Figure 2.5).

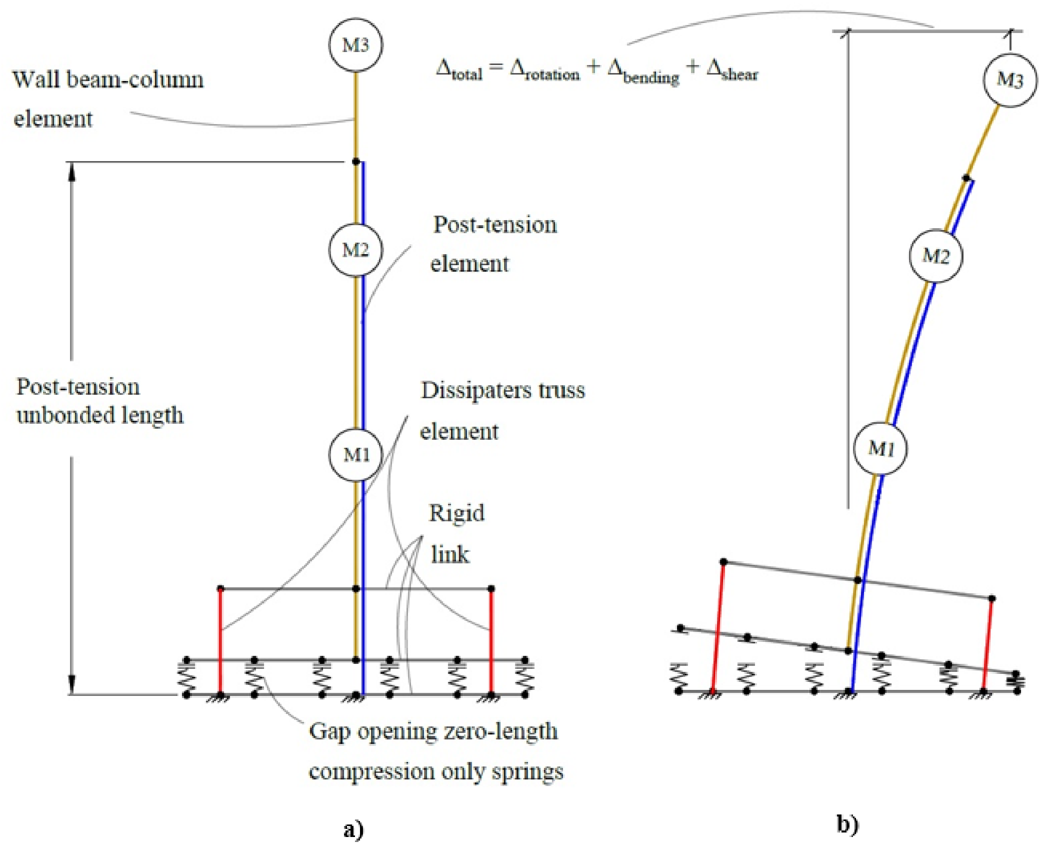


FIGURE 2.5 – Single wall multi-spring model overview : (a) initial state (b) deformed state

The springs are connected to a master node placed in the wall centerline using rigid-link elements. To allow rocking of the wall, the springs are modeled using a compression-only elastic-perfectly plastic hysteresis. Both types of wall panels are modeled as elastic beam-column elements and are connected to the master node of the multi-spring element. The post-tensioning bars are modeled as truss elements of the same unbonded length used in the analytical model. The bars are connected to the wall through rigid links at the anchorage point. The external tension-compression yielding dissipaters are also modeled as truss elements considering the Menegotto-Pinto hysteresis rule [MecKenna, 2011]. These truss elements are connected to the wall element using a rigid link beam and to the foundation using a fully restrained node. Figure 2.6 provides an overview of the complete multi-spring model in the initial (a) and in the deformed (b) states when the wall is rocking.

The numerical model has been calibrated through a static pushover analysis using a rotation displacement control from $\theta = 0$ to $\theta = 0.012$ rad. The axial stiffness of the multi-spring element was set to match the analytical moment-rotation as well as the neutral axis-rotation curves of the case study walls. Figure 2.6 shows the results of the numerical model calibration for case study building 1 in LVL, as an example.

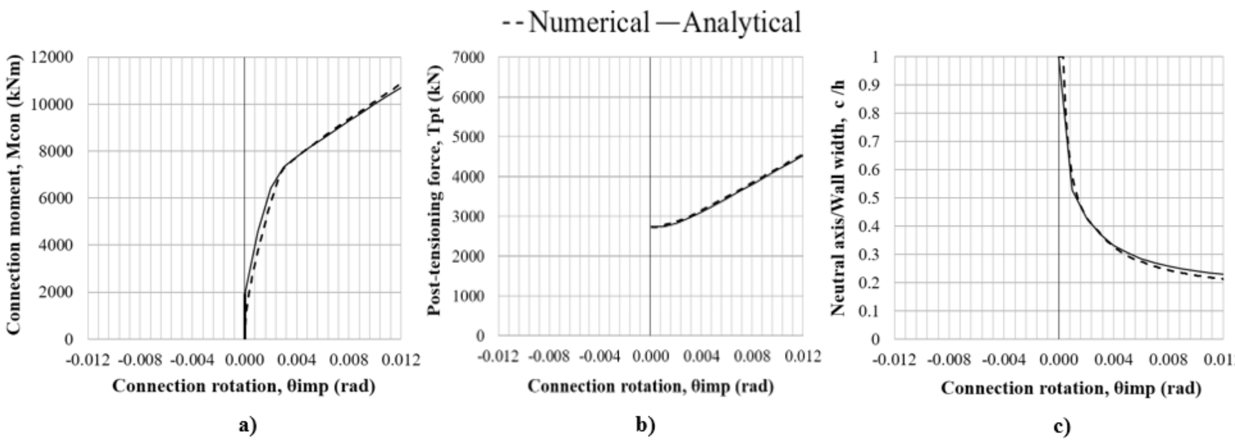


FIGURE 2.6 – Numerical model calibration for case study building 1 (LVL) (a) Connection moment (b) Post-tensioning force (c) Neutral axis position

To compare the behavior of single rocking segment systems and multiple rocking segment systems, NLTHA have been performed on multiple rocking segment numerical models developed and calibrated using the same concepts presented above. To allow rocking in the upper segments, additional multi-spring elements representing wood-to-wood contact between the panels have been introduced. To account for the reduced stiffness of the wood-to-wood contact, the axial stiffness of the compression springs has been calibrated considering a reduced modulus of elasticity. As for the single rocking segment models, the calibration has been done through a static pushover analysis using a rotation displacement control. The axial stiffness of each multi-spring element of the multiple rocking segment systems was first set to match the analytical moment-rotation, the post-tensioning force-rotation as well as the neutral axis-rotation curves in separate numerical models. Next, the complete multiple rocking segment models were built by stacking each segment with multi-spring elements and final static pushover analysis was performed on those complete models to ensure that they were behaving in accordance with the analytical studies.

The same selection of earthquakes chosen for the analyses of the single rocking segment systems has been used. The records spectra obtained from each ground motion have been scaled before running the NLTHA on the case study buildings.

2.5 Single Rocking Segment System

2.5.1 Analytical Study

An analytical study has been performed on single rocking segment systems with the walls made of LVL and of CLT to compare the sizes of panels, post-tensioning bars and external dissipaters needed to achieve the same moment-rotation behavior for both types of walls. In the study, the Modified Monolithic Beam Analogy (MMBA) has been applied for the moment-rotation analysis of post-tensioned rocking segments developed by Pampanin et al. [2001] and Palermo et al. [2004] and adapted to Pres-Lam systems by Newcombe et al. [2008]. This is an iterative procedure to calculate the forces in each element of the system during the rocking of the segment as a monolithic cantilever using strain compatibility. The second-order $P - \Delta$ effects are neglected assuming the panels having similar mass and enough rigidity that would not produce noticeable difference in the comparisons between the LVL and CLT systems.

The material properties of the system components used in the analytical study are presented in Table 2.2. The CLT properties are representative of a 9-ply panel composed of alternating longitudinal and transverse layers with a total thickness of 315 mm. Only the longitudinal layers of MSR-1950Fb lumber [CSA Group, 2009] were considered contributing in the design properties of the panels. To simplify the comparison between the systems, the LVL panels are built-up sections of 315-mm in thickness. High-strength steel is used for the post-tensioning bars designed to remain elastic, whilst mild steel is used for the external dissipaters designed to yield.

First, the LVL systems for each building have been designed with the smallest cross-section areas required to satisfy the following strength checks : panel resistance to in-plane bending, shear through thickness, compression, combined bending and axial compressive loads, post-tensioning bars yield strength, and dissipaters yield and ultimate strength. Then, the CLT systems have been designed and sized to achieve the same moment-rotation behavior as the LVL systems. Figure 2.7 shows the results for case study building 1. As Figure 2.7(a) shows, the same moment-rotation behavior was achieved for both types of panels. Figure 2.7(b) shows the difference in the post-tensioning forces and Figure 2.7(c) shows the ratio of the position of the neutral axis (c) to the width (h) of the wall.

Table 2.3 lists the details of the LVL and CLT systems with single rocking segments for all studied buildings including the the width (h), the thickness (b), the cross-section area (A_p) and the initial pre-stress (T_{pt0}) of the post-tensioning bars and the cross-section area of the external dissipaters (A_s). As can be seen from Table 2.3, the CLT segments need approximately 20% more width and 10% to 30% less initial pre-stress in the post-tensioning bars to obtain the same moment-rotation behavior as the LVL panels of the same thickness. These differences resulted in less cross-section area of post-tensioning bars and external dissipaters needed. The increases of the CLT wall width are mainly governed by the panel resistance to combined bending and axial compressive load. CLT is more sensitive to compressive loads than LVL, because the transverse layers in CLT have a negligible contribution to the panel resistance. For all the case study buildings, the neutral axis position is approximately the same for both types of panels, because the same moment-rotation behavior was achieved.

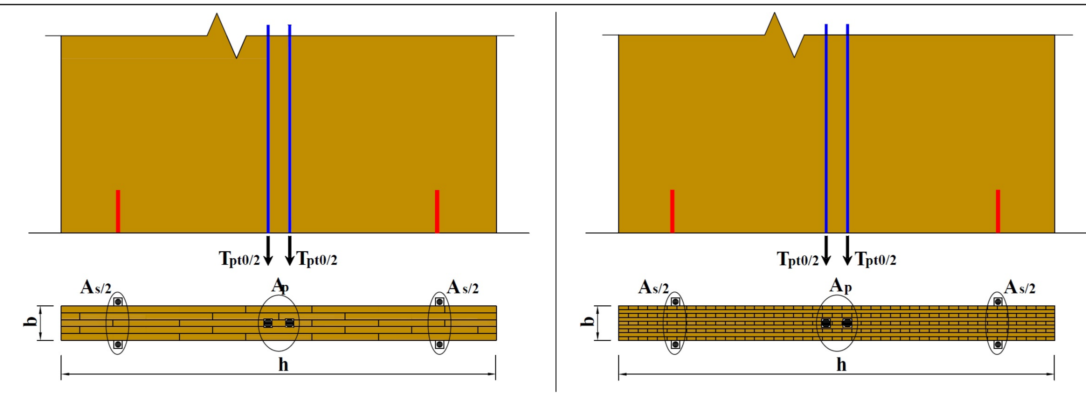
TABLE 2.2 – Material properties

CLT			**Post-tensioning bars**		
Bending strength of laminations (f_b)	28.2	MPa	Modulus of Elasticity (E_{pt})	170000	MPa
Shear through thickness strength of the panel (f_s)	1.6	MPa	Yield stress (f_{yp})	835	MPa
Compressive strength of laminations ($f_{c,0}$)	19.3	MPa	Ultimate strength (f_u)	1000	MPa
Equivalent MOE in major direction ($E_{0,eq}$)	6500	MPa	Post-yielding stiffness (r)	0.008	
Equivalent MOE in minor direction ($E_{90,eq}$)	4000	MPa	Mild steel yield strain (ε_{sy})	0.005	
Shear modulus (G_t)	654	MPa			
LVL			**Dissipaters**		
Shear modulus (G_t)	550	MPa	Modulus of Elasticity (E_s)	200000	MPa
Bending strength - in plane (f_b)	31.2	MPa	Yield stress (f_{ys})	300	MPa
Shear through thickness strength (f_s)	3	MPa	Ultimate strength (f_u)	450	MPa
Compressive strength (f_c)	45	MPa	Post-yielding stiffness (r)	0.008	
Modulus of Elasticity (E_t)	11000	MPa	Mild steel yield strain (ε_{sy})	0.0015	

Table 2.3 – Single rocking segment design results

ID	LVL						CLT					
	nWall	**h** (mm)	**b** (mm)	**A_p** (mm)	**T_{pt0}** (kN)	**A_s** (mm)	**nWall**	**h** (mm)	**b** (mm)	**A_p** (mm)	**T_{pt0}** (kN)	**A_s** (mm)
1	4	4000	315	2D65	2738	4D45	4	5000	315	2D60	1846	4D40
2	4	4200	315	2D65	3299	4D50	4	5200	315	2D60	2482	4D40
3	6	5300	315	2D70	4207	4D55	6	6600	315	2D65	3273	4D45
4	6	5600	315	2D70	3957	4D55	6	6800	315	2D65	3240	4D45
5	6	5600	315	2D70	4112	4D55	6	6800	315	2D60	3493	4D45
6	8	5600	315	2D70	4110	4D50	8	6500	315	2D55	2921	4D40
7	8	6000	315	2D65	3982	4D55	8	7200	315	2D60	3534	4D45
8	8	5800	315	2D70	4215	4D55	8	7200	315	2D60	3465	4D45

Drawings



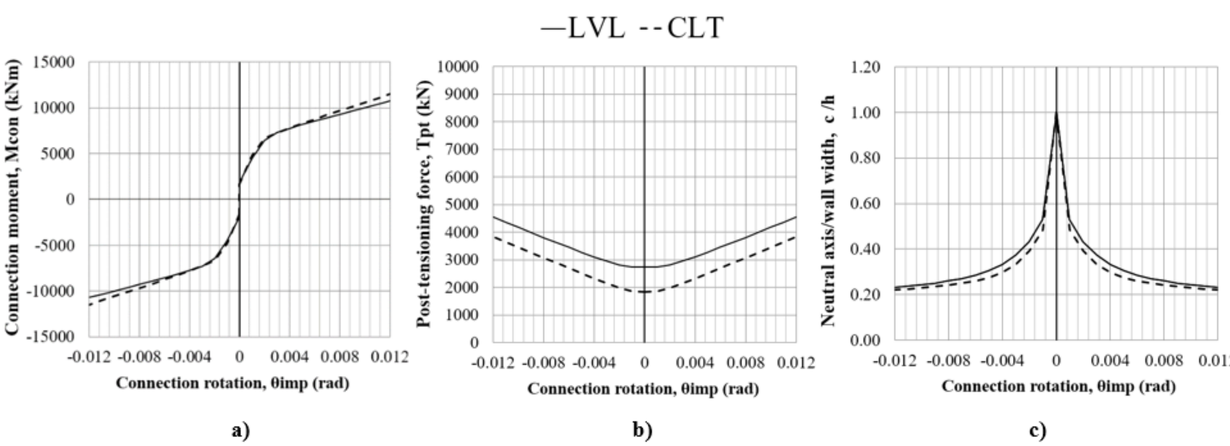


Figure 2.7 – Analytical results for case study building 1 (a) Connection moment (b) Post-tensioning force (c) Neutral axis position

2.5.2 NLTHA Results

The NLTHA have been performed for each case study building with LVL and with CLT walls. For brevity, the results of case study building 8 are reported in this paper. Similar outcomes have been observed for case study buildings 1 to 7. Figure 2.8 illustrates the shear and bending moment envelopes and the storey drift presented as a ratio of the average peak story drift to the building height for the buildings with LVL and with CLT walls. The curves obtained with the design moment distribution corresponding to the first mode response assuming a linear distribution of the forces along the height of the building are also shown for comparison. The building displacement profiles respect the limit imposed by NZS 1170.5 [NZS, 2004]. However, comparing the NLTHA envelopes and the design curves one can observe that the shapes of the envelopes are strongly influenced by the dynamic amplification at the upper floors. The amplification of the shear envelope at mid-height of the building, where the moment amplification is maximum, is less than in the lower and the upper storeys. The dynamic amplification occurs because of higher modes of vibration, which induce additional forces in the structure. For the case study building 8, the dynamic moment amplification factor at the building mid-height, is 2.67 for the LVL walls and 2.80 for the CLT walls.

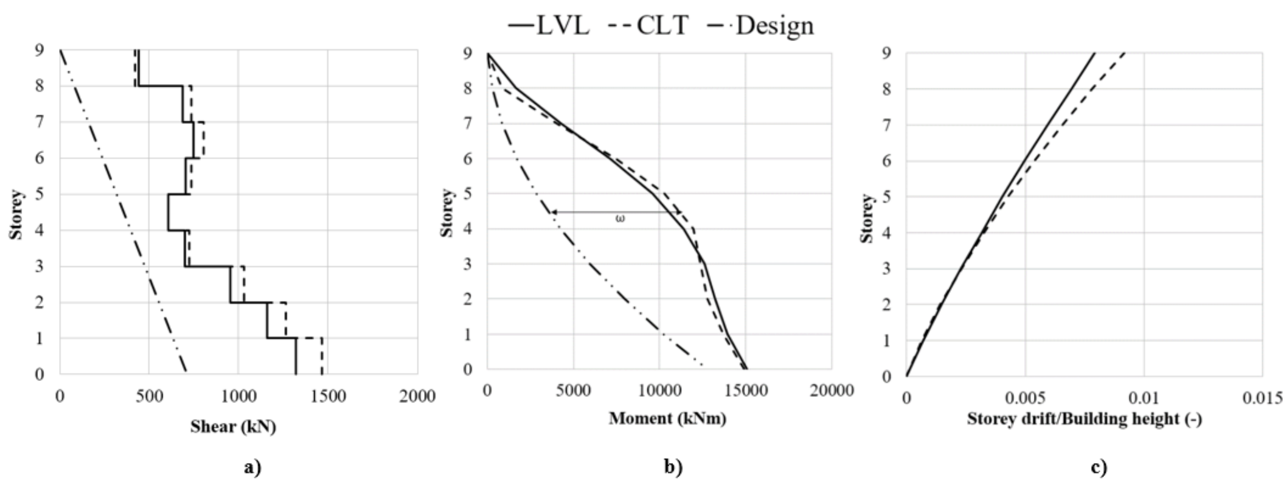


Figure 2.8 – NLTHA and design results for case study building 8 (a) Shear envelope (b) Bending moment envelope (c) Average peak drift

2.5.3 Connection Design

To ensure that the structure works as a single rocking segment system resisting high forces at the upper levels, large wall panels connected with strong and perfectly rigid horizontal construction joints between panels must be designed. To achieve it, steel glued-in bars or glued-in perforated plates or plates with multiple small-diameter fasteners are required, thus increasing the material and labour cost. Figure 2.9 shows an example of a nailed connection with steel plates for case study building 5 in LVL. To design this type of connection, the uplift force (F_{uplift}) at the end of the wall is determined from the maximum bending moment obtained from the NLTHA. The number and size of nails in the connection required to resist

F_{uplift} has been calculated in this project according to Eurocode 5 [CEN, 2004] and CSA O86 [CSA Group, 2009]. A total of 4000 rink-shank nails 6 mm in diameter 65 mm in length are required to join two panels, that is 1000 nails per steel plate on each side of the joint at each end of the panel. To check the horizontal shear resistance of the connection, a friction force ($F_{friction}$) generated by the post-tensioning was estimated assuming a friction coefficient of 0.25. It has been concluded that the $F_{friction}$ is sufficient to resist the maximum shear demand obtained from the NLTHA without considering the nails designed for the F_{uplift}. The steel plate of grade 300W has been designed according to CSA S16 [CSA Group, 2009] by checking the different modes of rupture in tension resulting in a 650-mm wide 10-mm thick plate. The slip modulus (K_u) of the connection has been evaluated according to Eurocode 5 [CEN, 2004] to confirm that a negligible 0.09-mm deformation of the connection would be observed under the design load if the nails are installed in pre-drilled holes. Figure 2.10 shows an example of this type of connection used at the base of the lateral-force-resisting system in a six-storey CLT building in Quebec, Canada.

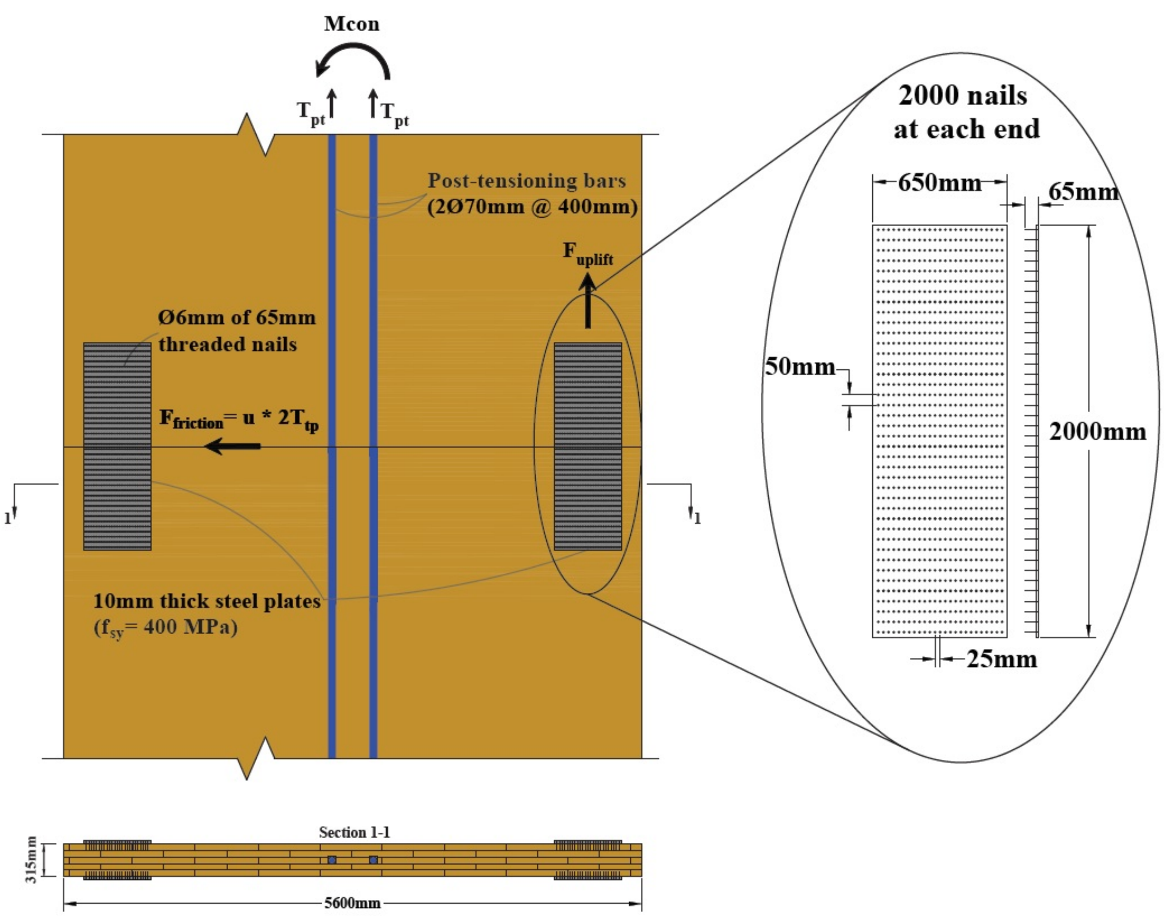


FIGURE 2.9 – Rigid connection design for single rocking segment system of case study building 5

FIGURE 2.10 – Example of a nailed CLT connection (photo courtesy of Nordic Structures)

2.6 Multiple Rocking Segment System

2.6.1 Analytical Study

To reduce the seismic demand along the wall, to simplify the connections between the wall panels and to reduce the construction costs, an alternative solution has been considered where the rigid connection shown above is replaced with a connection that allows a gap opening at the construction joints. In this case, the single rocking segment system turns into a multiple rocking segment system. To reduce the dynamic amplification, the dissipater elements are removed from the joints between the wall panels, which dissociates the segments and reduces the transferred moment. Due to the lower forces, it is possible to design a system with a smaller post-tensioning rod cross-section area and a lower initial pre-stressing. This concept was first studied with a simplistic rotational spring numerical model for concrete walls, which showed a significant reduction of structural forces in the upper levels because of higher modes of vibration [Wiebe & Christopoulos, 2009].

In multi-storey timber buildings, the construction joints can be introduced at every three storeys using 10 to 12-m long mass timber panels, which are readily available on the market and easy to transport from a factory to the construction site. To achieve the desired behavior of a multiple rocking segment system, the layout of the post-tensioning bars and the cross-section area of the wall panels must be carefully considered. Figure 2.11 shows schematically the configuration of a multiple rocking segment system with constant cross sections of panels along the height of the building and reduced cross-section areas and pre-stress forces of post-tensioning bars in the upper segment. The design moments at the gaps can be evaluated assuming a linear distribution of the bending moment along the height of the structure [Qureshi & Warnitchai, 2016]. The design base shear (V_b) and design base moment (M_b) obtained with the DBD and presented in Table 2.1 have been used to design the multiple rocking segment systems. The total deflection due the bending moment, shear, and gap opening needs to be within the permissible limit of the design drift ($\theta_d =$ of 1.2%) considering the reduced stiffness of the wood-to-wood bearing at the construction joints.

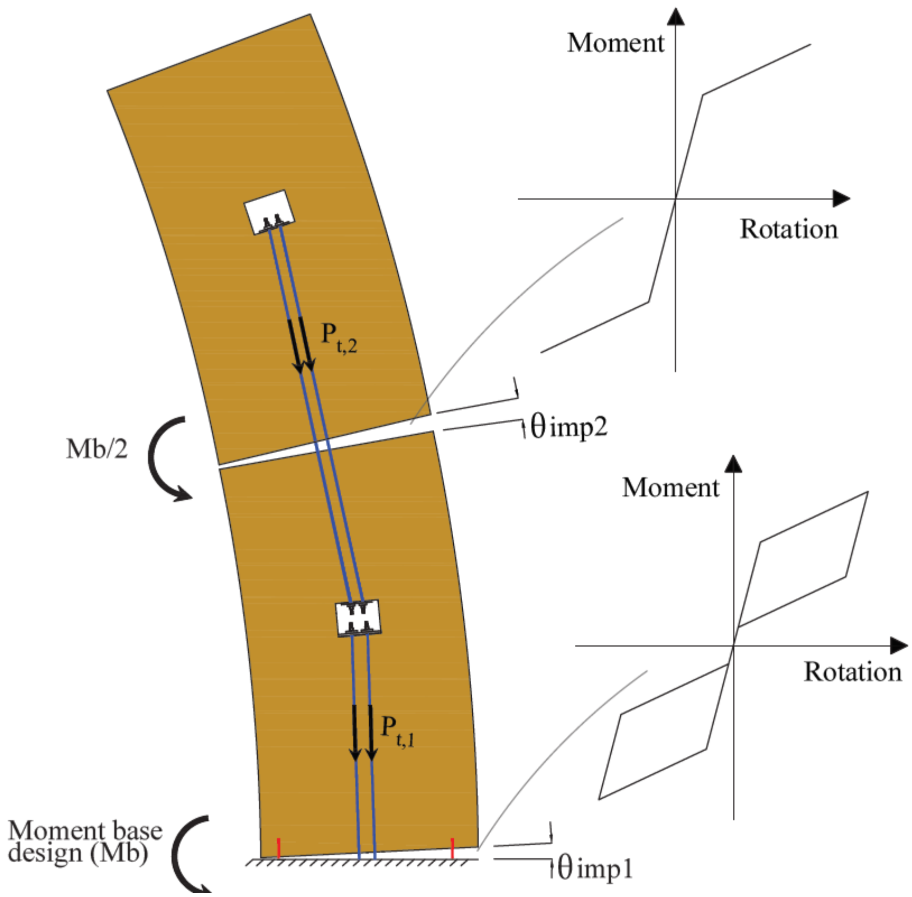


Figure 2.11 – Multiple rocking segment design

The design results of the multiple rocking segment systems with LVL and CLT walls for case study buildings 5 and 8 are presented in Table 2.4 including the width (h) and thickness (b) of the wall panels, the cross-section area (A_p) and the initial pre-stress (T_pt0) of the post-tensioning bars and the cross-section area of the external dissipaters (A_s). Like in the single rocking segment systems, the CLT segments need approximately 20% more width and 10% to 30% less initial pre-stress in the post-tensioning bars than LVL panels of the same thickness. Due to the lower seismic forces, the multiple rocking segment system may be designed with reduced cross section of panels in the upper levels, which would further reduce the cost of the building.

Table 2.4 – Multiple rocking segments design results

ID	Wall Section Storey	LVL					CLT				
		h (mm)	b (mm)	A_p (mm)	T_{pt0} (kN)	A_s (mm)	h (mm)	b (mm)	A_p (mm)	T_{pt0} (kN)	A_s (mm)
5	1 to 3	5600	315	2D70	3545	4D55	6800	315	2D70	3084	4D50
	4 to 6	4500	315	2D60	3082	Ø	5800	315	2D55	2102	Ø
8	1 to 3	5800	315	2D70	3457	4D60	7200	315	2D70	2694	4D50
	4 to 6	5200	315	2D60	3191	Ø	7000	315	2D50	1919	Ø
	7 to 9	4500	315	2D45	1659	Ø	6000	315	2D40	1028	Ø

2.6.2 NLTHA Results

The NLTHA have been performed for case study buildings 5 and 8 with LVL and with CLT walls with multiple rocking segments. For the sake of brevity, only the results for case study building 8 are reported in this paper. Figure 2.12 and Figure 2.13 illustrate the shear and bending moment envelopes and the storey drift presented as a ratio of the average peak story drift to the building height for the buildings with LVL and with CLT walls, respectively. The curves obtained with the NLTHA of the single rocking segment systems and with the design moment distribution corresponding to the first mode response are also shown for comparison. The shapes of the envelopes show a significant reduction in the shear forces and bending moments for the multiple rocking segment system relative to that with the single rocking segment. For the shear envelope, the reduction is localized in the upper storeys while the bending moment envelope is mitigated along the height of the building. The reduction is due to the gap opening between the upper and lower segments, which reduces the dynamic amplification (ω) caused by the higher modes present in the single rocking segment system. Some dynamic amplifications are still present in the upper storeys because of the flexible system and frequency content of the strong-motion records. The bending moment reduction (α) at mid-height of the building was 45% for the LVL walls and 42% for the CLT walls showing that it is possible to design the system with reduced cross-section areas of walls and post-tensioning bars and with lower initial pre-stressing, which would result in material and cost savings when using a multiple rocking segment concept.

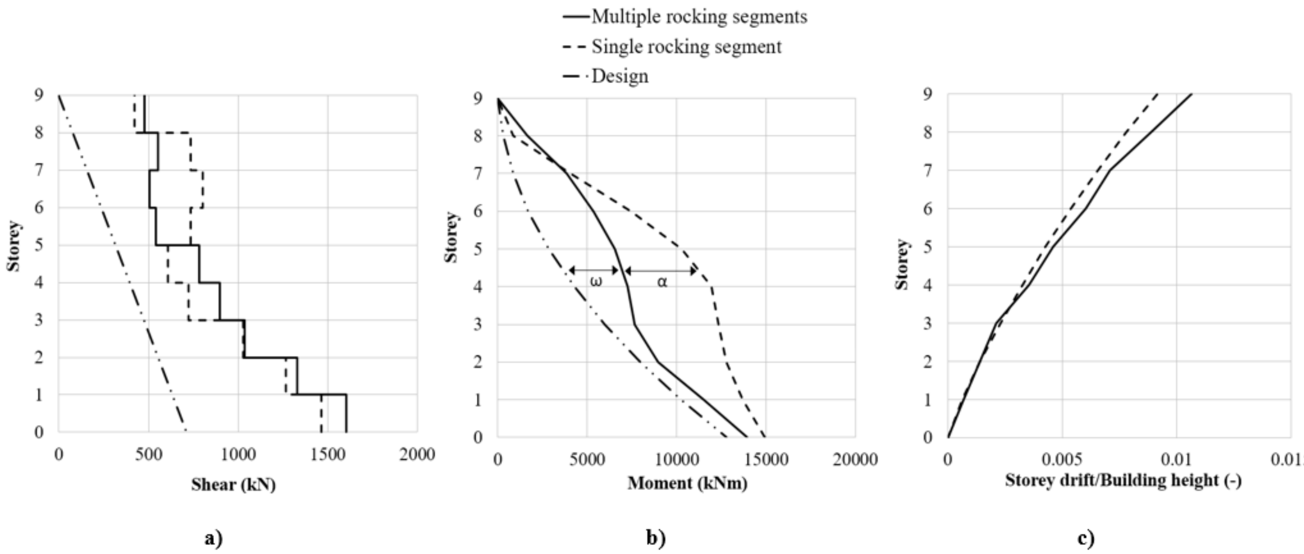


FIGURE 2.12 – NLTHA and design results for case study building 8 (LVL) (a) Shear envelope (b) Bending moment envelope (c) Average peak drift

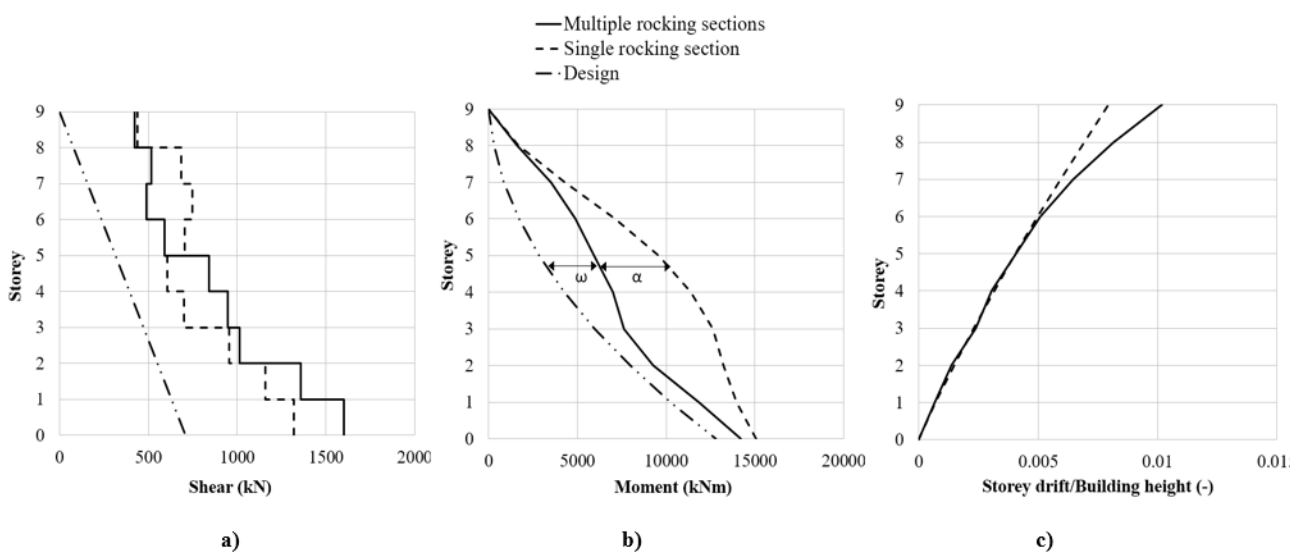


FIGURE 2.13 – NLTHA and design results for case study building 8 (CLT) (a) Shear envelope (b) Bending moment envelope (c) Average peak drift

2.6.3 Connection Design

In design of a multiple rocking segment system, it is important to address the detailing of the post-tensioning anchorage of the wall panel to avoid the localized damage during the seismic event. Figure 2.14 illustrates an example of such a connection. The post-tensioning bars are attached to a steel anchor plate with nuts and steel washer plates installed in an opening cut in the wall panel. The anchor plate is designed as a beam under uniformly distributed load resulting from the contact with the panel and supported by two rigid washer plates [Sarti, 2015]. A uniform stress distribution across the width of the anchor plate is assumed considering its high flexural stiffness relative to the timber bearing surface. The washer plates are considered rigid due to their very low slenderness ratio. To achieve uniform stress distribution in the timber bearing surface, the entire thickness of the wall panel is engaged in the contact with the plate. The area of the anchor plate is determined to satisfy the following condition : $0.4\sigma_r \geq \sigma_f$, where σ_r is the design strength on the timber-bearing surface and σ_f is the applied stress determined from the maximum force developed in the post-tensioning bar and uniformly distributed under the anchor plate.

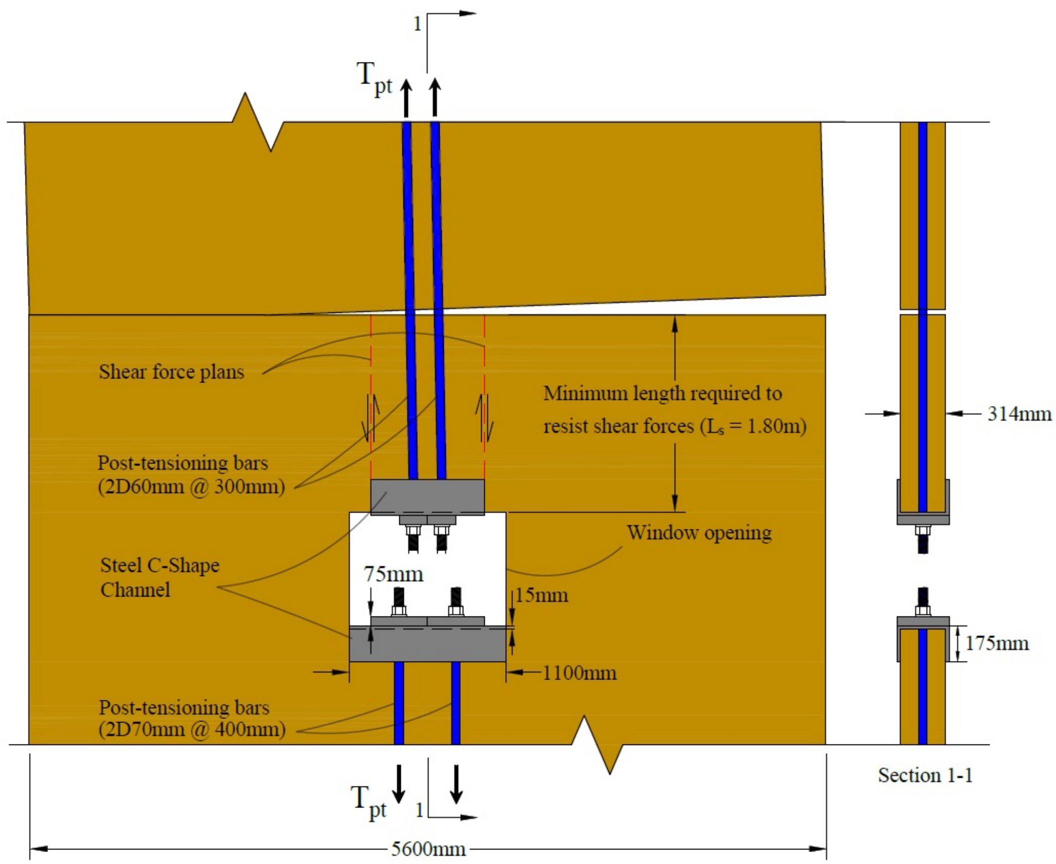


FIGURE 2.14 – Connection design for multiple rocking segment system of case study building 5

The required thickness of the anchor plate is calculated to resist the maximum bending moment developed under the washer plate. The shear and moment resistances of the reduced cross section of the wall panel at the opening must be checked. The post-tensioning forces induce high shear stress through the thickness of the wall panel starting at the corners of the opening, as shown in Figure 2.14. The length of the shear plane must be sufficient to ensure that the shear resistance of the panel is greater than the shear force. Also, the compressive resistance of the residual contact interface must be performed to avoid localized crushing of wood during the rocking motion of the upper wall segment. If needed, the edges of the wall panels can be reinforced to avoid localized crushing of wood.

2.7 Conclusion

The behavior of Pres-Lam wall systems with single and multiple rocking segments using mass timber panels has been analyzed and compared in this study. First, eight case study buildings with single rocking segment systems were designed with the walls made of LVL and of CLT panels aiming at the same moment-rotation behavior of both types of panels to compare the design requirements to each type of panel. Then, NLTHA analyses have been carried out on numerical models of Pres-Lam shear walls and connection detailing has been shown for single and multiple rocking segments. Based on the numerical analyses, the following conclusions have been made.

— Comparison between LVL and CLT single rocking segment systems demonstrated that the CLT panels needed approximately 20% wider cross section and 10% to 30% less initial pre-stress in the post-tensioning bars to obtain the same moment-rotation behavior as LVL panels. It also resulted in smaller post-tensioning bar and dissipater areas needed for CLT.

— NLTHA results for single rocking segment systems showed that the shear and bending moment envelopes were marked by a strong dynamic amplification up to a factor of 2.8 at mid-height of the structure. This amplification produced additional forces in the structures so that larger wall sections and strong rigid connections between the panels were required.

— The design of a rigid joint in a single rocking segment system resulted in a connection with thousands of nails. In addition to high material costs, it requires long installation time, thus high labor costs.

— NLTHA results for multiple rocking segment systems showed that the shear and bending moment envelopes were reduced 45% by allowing a gap opening between the adjacent wall segments. Due to the lower forces, it is possible to design a system with smaller cross-section areas lower initial pre-stressing of post-tensioning bars and simple connections between the wall segments resulting in material and labor cost savings.

— The connection detailing allowing gap openings in a multiple segment rocking system requires only a few steel plates and nuts, thus simplifying the design and installation resulting in a more economical solution.

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