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NUMERICAL SIMULATION FOR THE SEISMIC BEHAVIOUR OF MID-RISE CLT SHEAR WALLS WITH COUPLING BEAMS

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ABSTRACT: In this paper, an innovative type of mid-rise Cross Laminated Timber shear walls with coupling beams was designed. The 5-layer CLT panels were continuous along the height. Hold-downs and angle brackets were installed at the bottom of the panels. Coupling beams with energy dissipation devices were used to decrease the deformation and internal forces of the walls, providing adequate stiffness and strength. A numerical model was developed in OpenSees for a six storey prototype to investigate its seismic behaviour with different configurations. Strength degradation, stiffness degradation, and pinching effect were considered in the connection models. The structural performance was evaluated through a series of static and transient analyses. The simulation results indicated adequate lateral resistance and deformation capacity of this structural type. This study will lead to more application of large size CLT panels in multi-storey CLT buildings as lateral resistant systems.

KEYWORDS: Cross Laminated Timber, Mid-rise, Shear wall, Coupling beam, OpenSees

1 INTRODUCTION

Cross Laminated Timber (CLT) is an innovative wooden product. It is fabricated by bonding timber boards together with structural adhesives to produce a solid timber panel with each layer oriented crosswise to the next. A number of experimental studies on CLT panels has been conducted in Europe and Canada [1-4]. These tests indicated that, (1) CLT panels are relatively stiff, (2) the ductility and energy dissipation of the structures comes from the connections between wall panels, and between panels and base, dominated by rocking and slip mechanisms, and (3) connection details and panel sizes have significant effect on the lateral resistant performance of CLT panels.

Based on experimental work, numerical modeling of CLT panels was performed to predict the structural behaviour. Dujic et al. developed analytical models of CLT wall panels in the computer program SAP 2000 and verified them against test results [5]. Rinaldin et al. proposed a method for a more accurate evaluation of the energy dissipated by a subassembly made of cross laminated panels connected to each other and to the base using metal connectors [6]. Gavric modelled the connections with an advanced non-linear hysteretic spring to describe the cyclic behaviour of connections in ABAQUS finite element software package as an external subroutine [7]. Pei et al. developed and calibrated a simplified numerical model to predict the reverse cyclic behaviour of CLT walls with test results [8].

Coupling beams are commonly used components in reinforced concrete shear wall structures. They are defined as the connecting beams between walls with a span-depth ratio less than five. Under seismic or wind load, bending of the shear walls causes deformations at the end of the coupling beams. The forces of the beams constrain the coupled walls in turn. Thereby, the internal forces and deformation of the shear walls are decreased, providing a better structural behaviour. The detailing between beams and walls significantly influence the structural performance of the buildings. Improper design of coupling beams may fail to meet the lateral strength and stiffness requirements of the CLT shear walls. However, this has seldom been studied.

In this paper, a case study of a six storey CLT shear walls with coupling beams was carried out to evaluate the seismic behaviour of such structural type. Design of hold downs and angle brackets were calibrated to match expected structural performance. Hold downs and angle brackets were modelled as *ZeroLength* elements with a *Pinching4* hysteresis type in OpenSees [9]. The beam-and-wall connections were modelled as *TwoNodeLink* element with a *Steel01* hysteresis type in Opensees. The periods and modes, global responses and local responses of the

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structure were discussed through a series of static and dynamic analyses. Optimal design for such structure type was selected though targeting performance evaluation. Design recommendations were given based on the analytical results.

2 DESCRIPTION OF THE PROTOTYPE

2.1 GEOMETRY AND LOADS

The prototype of cross laminated timber shear walls with coupling beams has six storeys with the elevation shown in Figure 1. The walls and coupling beams were made of 175 mm thick 5-layer CLT panels. The walls were continuous in the long direction prefabricated in the factory. The panels were connected to the base with hold-downs on the sides and angle bracket in the centre (Figure 2). Coupling beams were adopted to connect the two walls with steel plates. The steel plates with dowels will transfer lateral forces. The low yielding dampers in the centre of the coupling beams will undergo shear deformation during the earthquake to dissipate energy (Figure 3).

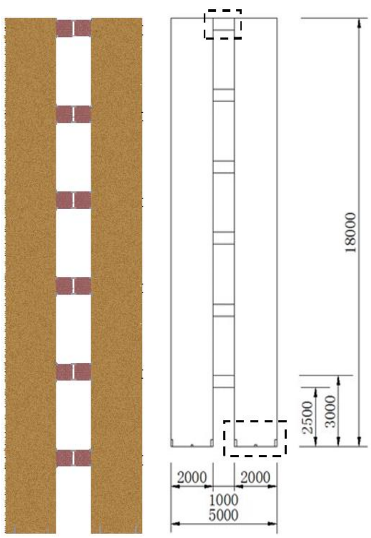


Figure 1: *Elevation of the shear wall with coupling beams*

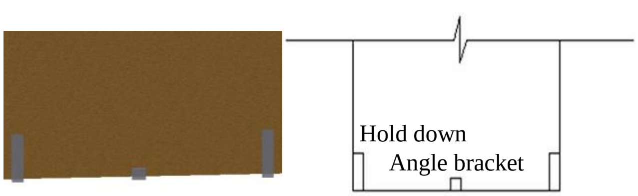


Figure 2: *Configurations of hold downs and angle brackets*

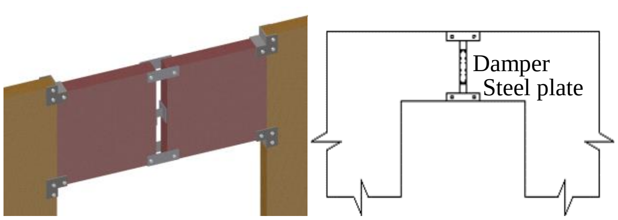


Figure 3: *Configurations of dampers*

It was assumed that the structure was located in Vancouver, B.C. on Site Category C. The hazard used in this design were S_a = 0.96 g at 0.2 s, 0.64 g at 0.5 s, 0.33g at 1.0 s and 0.17 g at 2.0 s. The shear walls held up an area of 40 m^2 for each storey. The dead load and live load were 1.5 kPa and 2.5 kPa, respectively. The density of CLT panel is 480 kg/m^3.

2.2 CONNECTION DESIGN

The total seismic weight of the building was 514.7 kN. The Equivalent Static Force Procedure according to 2010 NBCC [10] was followed to calculate the seismic design demand for the shear walls.

$$V = S(T_a) M_V I_E W / (R_d R_0) \qquad (1)$$

The fundamental period of the 6-storey shear walls determined using the NBCC formula for shear wall structures was T_a = 0.437 s. The value for the over-strength modification factor R_o for this study was chosen to be 1.5 for CLT structures. The value for the ductility modification factor R_d was chosen as 1.5 conservatively. The building was assumed to have an importance factor of 1.0. The base shear demand was 124.04 kN. The shear forces were considered taken by the angle brackets only. Based on previous experimental work [6, 11], two angle bracket connections (2A) consisted of "SIMPON StrongTie bracket 90×48×3.0×116 mm (BMF 07716) and 18 Spiral nail 16d*-3 1/2" (Figure 4) were installed at each "angle bracket" location in Figure 2.

Figure 4: *BMF 07716 (Shen et al. [11])*

To resist the overturn moment, two SIMPON HTT22 (Figure 5) connected to the panel with 14 annular ringed shank nails 4×60 type Anker and to the base with a 16 mm diameter bolt (2H) at each "hold down" location in Figure 2. Low yielding steel plates [12] were chosen as dampers (Figure 6). One such damper (1D) was installed at the centre of each coupling beam. Steel plates with dowels adopted to transfer lateral forces were considered as rigid links in the horizontal direction. This preliminary design was denoted as 2A2H1D. It will be optimized by later parameter studies.

Figure 5: *SIMPON HTT22 (Rinaldin et al. [6])*

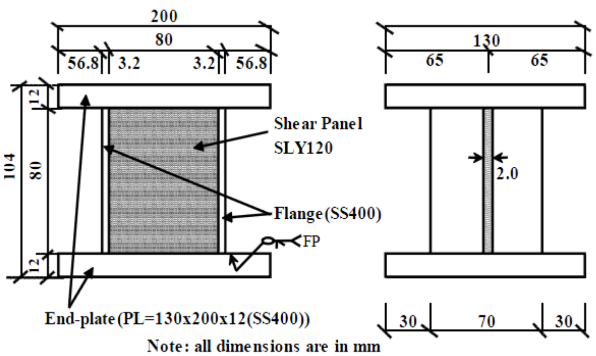


Figure 6*: Low yielding damper configurations (Abebe et al. [12])*

2.3 KINEMATIC MECHANISM

Figure 7 gives the kinematic mechanism of the rocking of the shear walls with coupling beams.

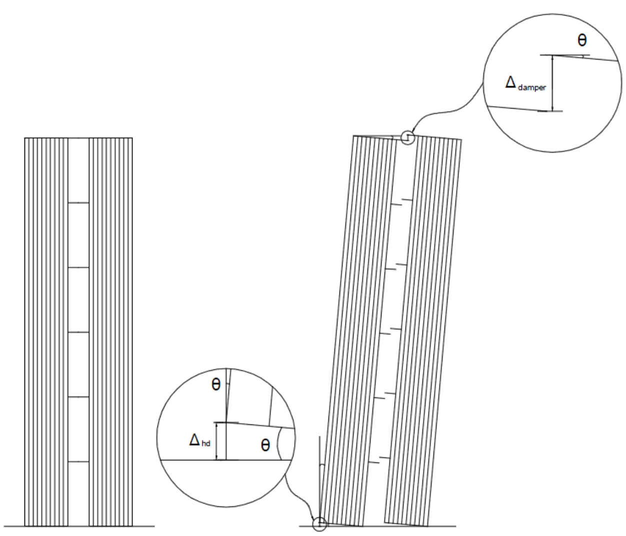


Figure 7*: Kinematic mechanism of rocking shear walls with coupling beams*

To prevent the failure of the hold down, which will lead to the overturning of the structure, the drift ratio of the structure has to be controlled within 1.775%. This indicates the relatively low ductility of the structure using these traditional connectors. Such deficiency should be avoided by adopting components with recentering capacity, such as high strength rods or posttensioned tendons in the future. In this study, the structure was designed to satisfy this drift ratio limitation. Here, we required that the maximum limitation should not exceed 1.5% under the maximum considered earthquake.

3 NUMERICAL MODELING APPROACH

3.1 MODEL ASSUMPTIONS

The numerical model was developed in the finite element computer program OpenSees. A few assumptions were adopted to simplify the analysis, (1) hold-downs and dampers can only take vertical forces and angle brackets only take horizontal force, (2) CLT was considered as an elastic material, (3) the shear deformation of coupling beams only happens at the dampers.

3.2 MODEL DISCRIPTION

In OpenSees, CLT panels were modelled with *quad* element. Hold downs and angle brackets were modelled by *ZeroLength* element with *Piching4* material. This material considers the strength degradation, stiffness degradation, and the pinching effect of the connections. Opensees provided an *elastic No-tension* material which could be applied to the additional *ZeroLength* elements to model the high compression stiffness at the base. The dampers were modelled by *TwoNodeLink* element with *Steel01* material in the vertical direction, and rigid in the horizontal direction. The models were calibrated based on existing experimental data [6, 11].

3.2.1 Connection models

Pinching4 model used for the angle brackets and hold downs is composed by piecewise linear curves, which represents a ‘‘pinched’’ load-deformation relationship and accounts for stiffness and strength degradation under cyclic loading [13].

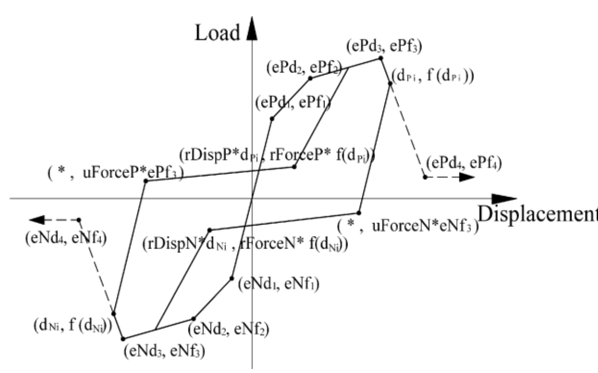


Figure 8: *Pinching4 model (Shen et al. [11])*

As for the model in Figure 8, the piecewise linear curves involving 16 parameters (ePd1, ePf1, ePd2, ePf2, ePd3, ePf3, ePd4, ePf4, eNd1, eNf1, eNd2, eNf2, eNd3, eNf3, eNd4, eNf4) are used to define the response envelope. Two unload-reload paths and pinching behavior are defined with 6 parameters (rDispP, rFoceP, uForceP, rDispN, rFoceN, uForceN). rDispP and rDispN respectively refer to the pinched ratio of the deformation at which reloading or unloading occurs to the historic deformation demand of each cycle. rForceP and rForceN individually indicate the pinched ratios of the forces corresponding to the historic deformation demand of each cycle under reloading and unloading. uForceP and uForceN represent the pinched ratios of strengths under reloading and unloading, respectively.

There are 16 parameters to control increasing unloading stiffness degradation, accelerated reloading stiffness degradation and strength degradation under cyclic loading. The damage indices dki, ddi, and dfi are assumed to be a function of displacement history and energy accumulation when setting the damage type as “energy”.

For *Pinching4* model, the current reloading stiffness is defined based on the current loading history. It can be used for an asymmetry hysteretic behavior because of respective definition of the positive and negative hysteretic curves. Connection failure happens when displacement demand exceeds the envelope curve defined before. The calculations continue to run and the failure segment of the envelope curve is characterized by the horizontal line.

The parameters of *Pinching4* model for angle brackets were calibrated by Shen et al. [11] (Figure 9) while the parameters for the hold downs were calibrated in this study (Figure 10) based on Rinaldin et al.`s experiment results [6].

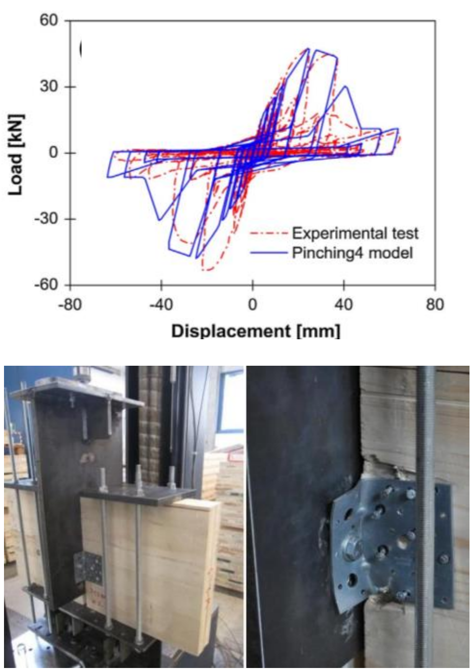


Figure 9: *Pinching4 model for angle brackets and angle bracket slip (Shen et al. [11])*

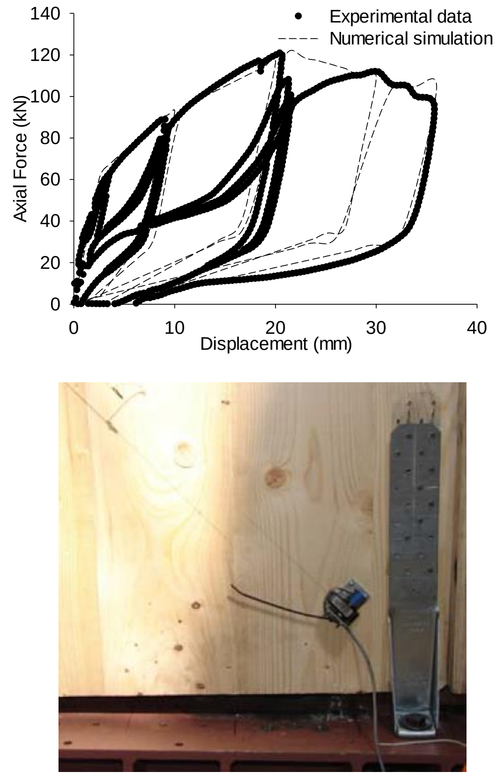


Figure 10: *Pinching4 model for hold downs and hold down uplift (Rinaldin et al. [6])*

3.2.2 Model parameters

Table 1-3 shows the parameters of the component models in OpenSees.

Table 1: *parameters of the hold down models*

Tag and Material	Parameters	
Hold down (1H)	ePf1 63.6 kN	uForceN 0.23
uniaxialMaterial	ePd1 2.11 mm	gK1 0
Pinching4	ePf2 82 kN	gK2 0
	ePd2 9 mm	gK3 0
	ePf3 121 kN	gK4 0
	ePd3 20.5 mm	gKLim 0
	ePf4 105 kN	gD1 0.97
	ePd4 35.5 mm	gD2 0
	eNf1 -63.6 kN	gD3 0
	eNd1 -2.11 mm	gD4 0
	eNf2 -82 kN	gDLim 0.05
	eNd2 -9 mm	gF1 0
	eNf3 -121 kN	gF2 0
	eNd3 -20.5 mm	gF3 0
	eNf4 -105 kN	gF4 0
	eNd4 -35.5 mm	gFLim 0
	rDispP 0.85	gE 1
	rForceP 0.46	dmgType Energy
	uForceP 0.23	
	rDispN 0.85	
	rForceN 0.4	

Table 2: *parameters of the angle bracket model*

Tag and Material	Parameters	
Angle bracket (1A)	ePf1 19.5 kN	uForceN 0.05
uniaxialMaterial	ePd1 2.15 mm	gK1 0
Pinching4	ePf2 44.89 kN	gK2 0
	ePd2 8 mm	gK3 0
	ePf3 49.45 kN	gK4 0
	ePd3 20 mm	gKLim 0
	ePf4 6.38 kN	gD1 0.95
	ePd4 60 mm	gD2 0
	eNf1 -19.5 kN	gD3 0
	eNd1 -2.15 mm	gD4 0
	eNf2 -44.89 kN	gDLim 0.1
	eNd2 -8 mm	gF1 0
	eNf3 -49.45 kN	gF2 0
	eNd3 -20 mm	gF3 0
	eNf4 -6.38 kN	gF4 0
	eNd4 -60 mm	gFLim 0
	rDispP 0.5	gE 1
	rForceP 0.3	dmgType Energy
	uForceP 0.5	
	rDispN 0.05	
	rForceN 0.3	

Table 3: *parameters of other component models*

Tag and Material	Parameters	
CLT *nDMaterial ElasticIsotropic*	E 9500 Mpa	v 0.46
Coupling Beam *ElasticBeamColumn*	E 10^6 Mpa G 10^6 Mpa A 10^5 mm^2	J 2×10^8 mm^4 I_y 10^8 mm^4 I_z 10^8 mm^4
Damper (1D) *uniaxialMaterial Steel01*	F_y 12.99 kN E 5.19 kN/mm b 0.05 a1 0	a2 1 a3 0 a4 0
Rigid *uniaxialMaterial Elastic*	E 10^8 kN/mm	
Compression *uniaxialMaterial ENT*	E 8×10^5 kN/mm	

4 NUMERICAL MODELING ANAYSIS

In this preliminary study, push over analyses were firstly conducted for seven configurations of connectors (2A2H1D, 2A2H0.5D, 2A2H2D, 2A1H1D, 2A3H1D, 1A2H1D, 3A2H1D) to find an optimal design. The calibrated design was then subjected to a set of time history analyses of ten ground motions at three hazard levels (Serviceability with a probability of 50% in 50 years, life-safety with a probability of 10% in 50 years, and maximum considered earthquake with a probability of 2% in 50 years). The ground motions were selected from NEES strong ground motion database [14] and scaled to match the target spectra at the range of 0.2 to 1.5 times of the fundamental period of the structure. Figure 11 shows the scaled spectra under three hazard levels.

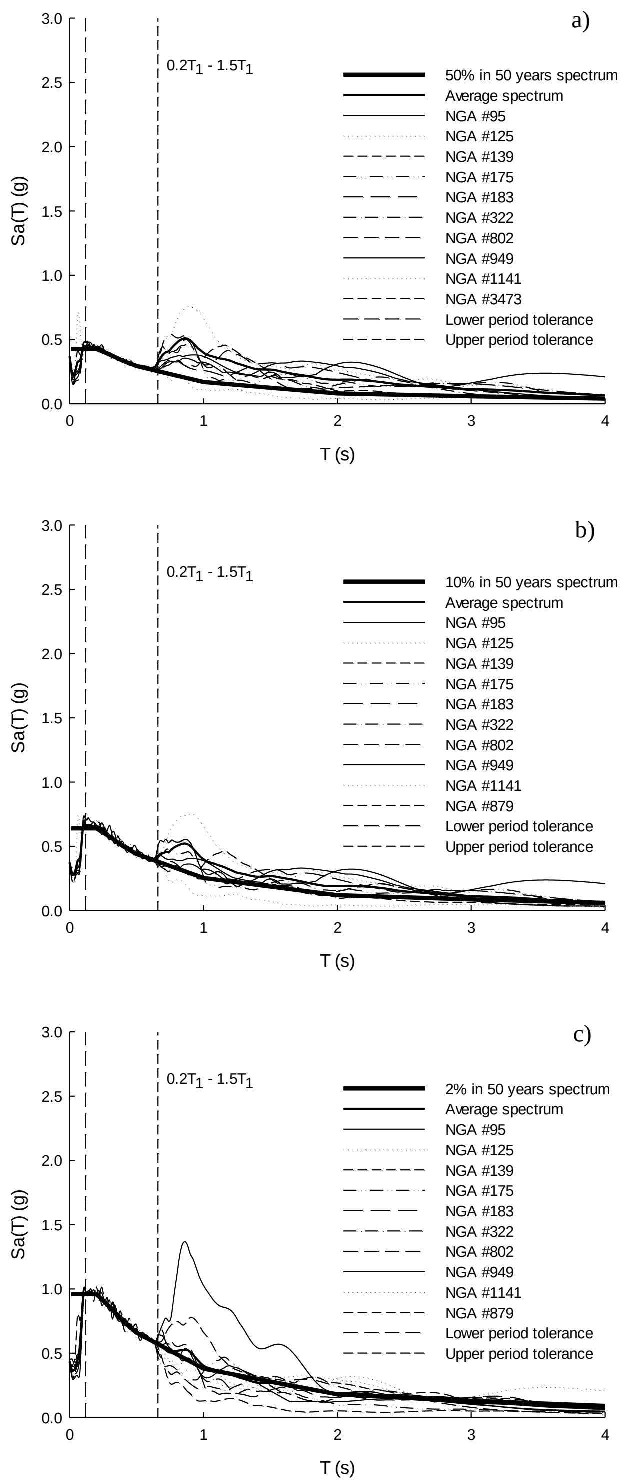


Figure 11: *Scaled ground motion spectra for three hazard levels a) 50% in 50 years; b) 10% in 50 years; c) 2% in 50 years*

5 RESULTS AND DISCUSSION

5.1 MODE RESPONSE

The first two periods and modes of the structure with 2A2H1D configuration were shown in Figure 12. Structures with other configruations have different periods but very close mode shapes. The indicated that the CLT panels are relative rigid and the connectors govern the structural behavior.

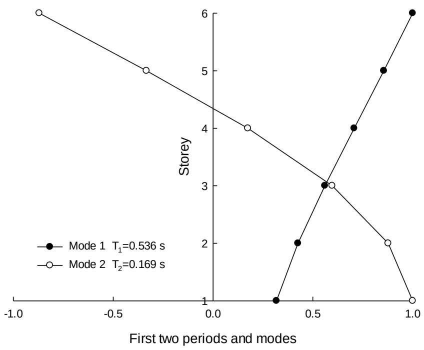


Figure 12: *First two periods and modes of the model*

5.2 PUSHOVER ANALYSIS

A parameter study was carried out by a series of push over analyses of shear walls with different configuration of angle brackets (A), hold downs (H) and dampers (D) to evaluate their performances. Figure 13 shows the push over curves of different structures.

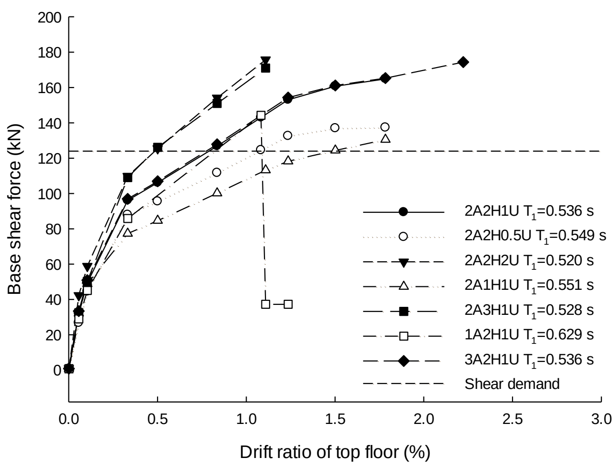
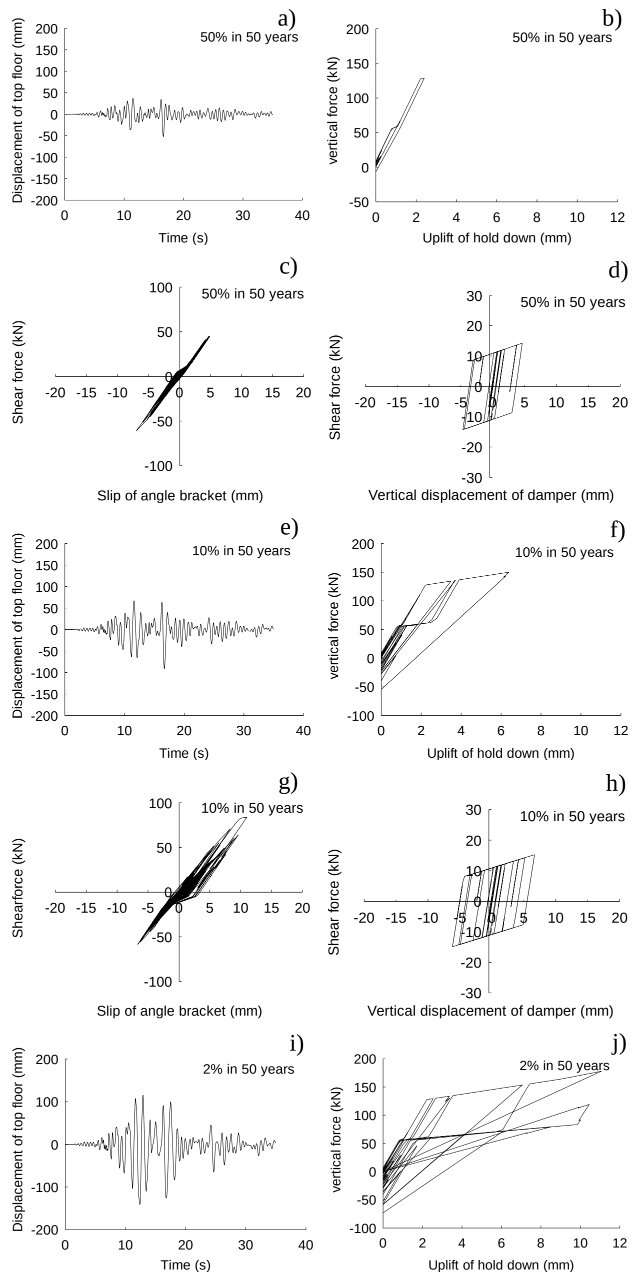


Figure 13: *Push over curves of different structures*

1A2H1D failed with poor shear at 1.06% drift ratio. 2A2H2D and 2A3H1D failed of high rigidity before 1.39% drift ratio. 2A1H1D and 2A2H0.5D had enough ductility but relatively low strength. 3A2H1D had the same performance as 2A2H1D but wasted the angle brackets. 2A2H1D was the optimal design for this case.

5.3 TIME HISTORY ANALYSIS

Thirty time history analyses (ten records and three hazard levels) were conducted for the structure with configurations of 2A2H1D. Four Engineering Demand Parameters (EDPs), the maximum drift ratio of the structure (DR), the maximum uplift of the hold downs (UP), maximum slip of the angle brackets (SL), and the maximum shear deformation of the dampers (DE), were used to evaluate its seismic behaviour. Figure 14 shows the displacement time history of the top floor, and the relationships between force and deformation of the connectors under three different hazard levels for ground motion #949.



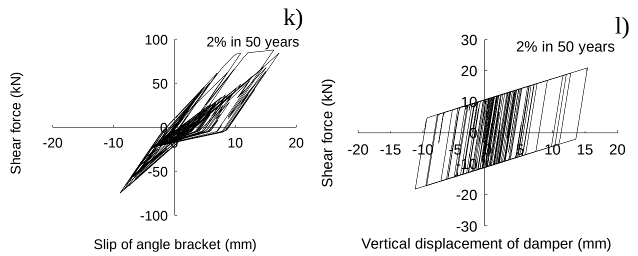


Figure 14: *Displacement time history of the top floor, and force-deformation relationship of the connectors under three different hazard levels for ground motion # 949*

For the ground motion record # 949, the damper had started to dissipate energy while hold downs and angle brackets almost remained elastic under 50% in 50 years hazard level. Under 10% in 50 years hazard level, all connectors entered inelastic during the seismic excitation, but still reserved considerable strength and deformation capcity. Under 2% in 50 years hazard level, the whole structure indicated high nonlinearity. However, none of the connecters showed failure from the analysis.

Figure 15 shows the dynamic responses of the building under different records for 2% in 50 years hazard level. The drift ratio under individual records, the mean drift ratio, and the mean plus standard deviation (stdv.) are given in Figure 15 a). The shape of the curves were nearly linear at above stories. The maximum mean and mean plus stdv. values were 0.86% and 1.21%, respectively. These exhibit the relatively high stiffness of the shear walls. The slope of drift at first floor was smaller than upper ones. This reflects the deformation of the connectors at the base. All the connectors started to dissipate energy at a relatively small deformation. Figure 15 b) gives the maximum uplift of the hold downs. The mean plus standard deviation was below the deformation of the hold downs under maximum capacity of shear force. It means that the connectors had satisfying redundancy to undergo more uplift. Figure 15 c) shows the maximum slip of the angle brackets at the base. Some angle brackets had already reached their maximum capacity of shear force. But the mean plus standard deviation was still far from the ultimate deformation limit. Figure 15 d) is the rotation of the dampers under different records. The deformation of these dampers was close to their ultimate deformation capacity. They could be easily replaced after the earthquake if failure occurs. It is noted that not only the structure experienced a quite small maximum drift ratio, the residual deformation after the earthquake was also negligible.

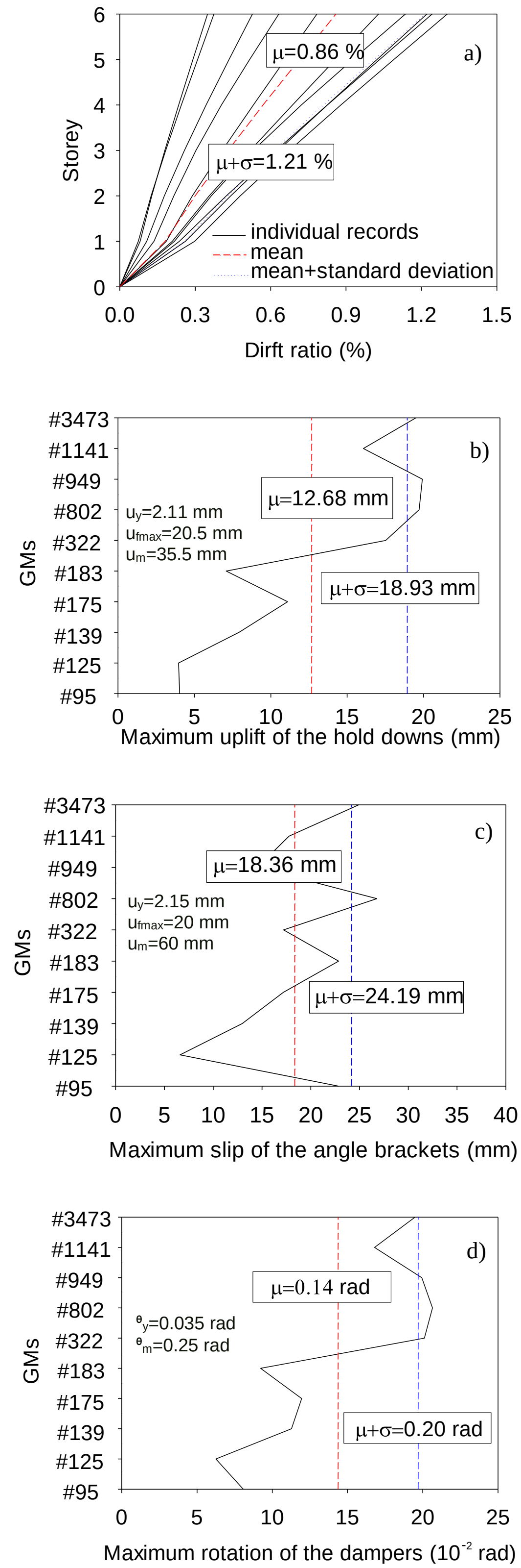


Figure 15: *EDPs under different records for 2% in 50 years hazard level*

6 CONCLUSIONS

Innovative mid-rise CLT shear walls with coupling beams were designed in this study. Numerical modelling, structural analysis and performance evaluation of the proposed structural type with different configurations were conducted. The results implied that, in mid-rise CLT shear walls with coupling beams, the shear resistance is controlled by the angle brackets, while the deformation capacity is controlled by the hold downs and coupling beam connections. As major lateral resistance members in the building, the relatively high shear and uplift demand for CLT shear walls is significant in configuration design. Due to the relatively rigid property of CLT panels, hold downs require much more vertical stiffness than traditional ones used in common wooden buildings. It has been noted that using conventional hold down connections only may not be able to provide satisfying ductility and deformation capacity for the structure. The risk of overturning of the structure should be avoided by adding additional components that can provide uplift resistance, such as high strength rods and posttensioned tendons.

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