Ductility factor validation for CLT walls with bolted hold-downs: preliminary analyses and observations

B. Moerman, M. Li, A. Palermo
University of Canterbury, Christchurch

T. Smith
PTL Structural Consultants, Christchurch

A. Liu
BRANZ, Wellington

ABSTRACT

High-capacity cantilever CLT walls can be created with bolted hold-downs to resist earthquake loading. This study briefly presents the experimental results and cyclic behaviour of the representative test specimen. Then prototype buildings of 3 and 6-storeys in Christchurch, NZ are designed using an assumed ductility factor of three following the equivalent static method in NZS 1170.5. The cantilever walls were then subjected to nonlinear time history analyses using 12 spectrum-compatible ground motions to evaluate the validity of the selected ductility factor and a deflection amplification factor k_{dt} which is currently proposed as one method of accounting for pinched hysteretic behaviour in the calculation of maximum displacements. Based on the results, it was found that the ductility factor was appropriate but the k_{dt} factor tended to the conservative side for the 3-storey building and overestimated the deformations in the 6-storey building. Additionally, a severely damaging response was observed for one of the two ground motions used from the 2010 Darfield earthquake due to the acceleration spike at long periods evident in the recorded motion and may present similarly excessive damage for other pinched hysteretic systems.

1 INTRODUCTION

Cantilever cross-laminated timber (CLT) walls for lateral seismic force resisting systems in buildings are becoming increasingly popular as these elements become more economical and more owners consider the sustainability of their buildings. Conventional CLT walls are constructed with base connections made of light gauge steel plate and large groups of nails or screws (Rothoblaas 2020). However, due to their relatively low strength and stiffness, they can result in limiting the full potential of CLT which possesses inherently high in-plane strength and stiffness. Therefore, other fasteners like self-tapping screws, dowels (Ottenhaus et al. 2018), or bolts are viable solutions for CLT walls with greater capacities.

CLT is a new engineered timber material, therefore they are not currently part of the NZ timber design standard NZS 3603-1993 (New Zealand Standard 2005) or the proposed draft of the upcoming replacement standard AS/NZS 1720. Therefore, the commonly used equivalent static method cannot be used without the designers completing a higher degree of analysis (e.g. nonlinear time history) while following an alternative means to code compliance and receiving a competent peer review. The critical factor in the equivalent static method is the ductility factor, μ, which is the assumed ratio of peak to yield displacement and is used to find the force reduction factor k_{μ}.

While cyclic wall testing and shake table experiments have been completed (as shown in (Pei et al. 2016)), there is a lack of consensus on an acceptable ductility factor to use for the design of CLT wall systems. Tannert (2019) and Pei et al. (2013) proposed the use of R_d=2 and R_o=1.5 for use in the National Building Code of Canada (National Research Council Canada 2015) ($R_d \sim k_{\mu}$ and $R_o \sim 1/S_p$ in NZS 1170.5 (New Zealand Standard 2016)). In 2018, Follesa et al. (2018) proposed a behaviour factor (q) of 2 for cantilever CLT walls as part of a draft for an updated version of Eurocode 8 (European committee for Standardization (CEN) 2003) ($q=k_{\mu}$ in NZS 1170.5). More recently, Faggiano et al. (Faggiano et al. 2022) proposed design rules for an Italian annex to Eurocode 8 with a behaviour factor of three for CLT walls with sufficient aspect ratios, fastener slenderness, and adequate capacity protection for brittle failure mode.

This study briefly presents cyclic wall test results and discusses the observed hysteretic behaviour of a cantilever CLT shear wall with bolted hold-downs. A prototype building is then designed using the equivalent static method with an assumed ductility factor of three and is subjected to nonlinear time history analyses. Finally, the results of the analyses are presented and discussed.

2 SEISMIC BEHAVIOUR

To investigate the cyclic behaviour of CLT walls with bolted hold downs, a set of three specimens with different aspect ratios were tested cyclically at the University of Canterbury’s Structural Engineering Laboratory. The tallest experiment is shown in Figure 1a with bolted hold-downs using 4-⌀20mm bolts (Figure 1b) made by mild steel round bar threaded at each end. The bolts experienced a ductile failure mode by bending at four hinge locations, elongating, and crushing timber locally, as shown in Figure 1c and d. Capacity design, using an overstrength factor of three, was used to protect the non-ductile elements in the system, including the loading clamp, hold-down fittings, shear key, CLT wall panel, and steel foundation fittings.

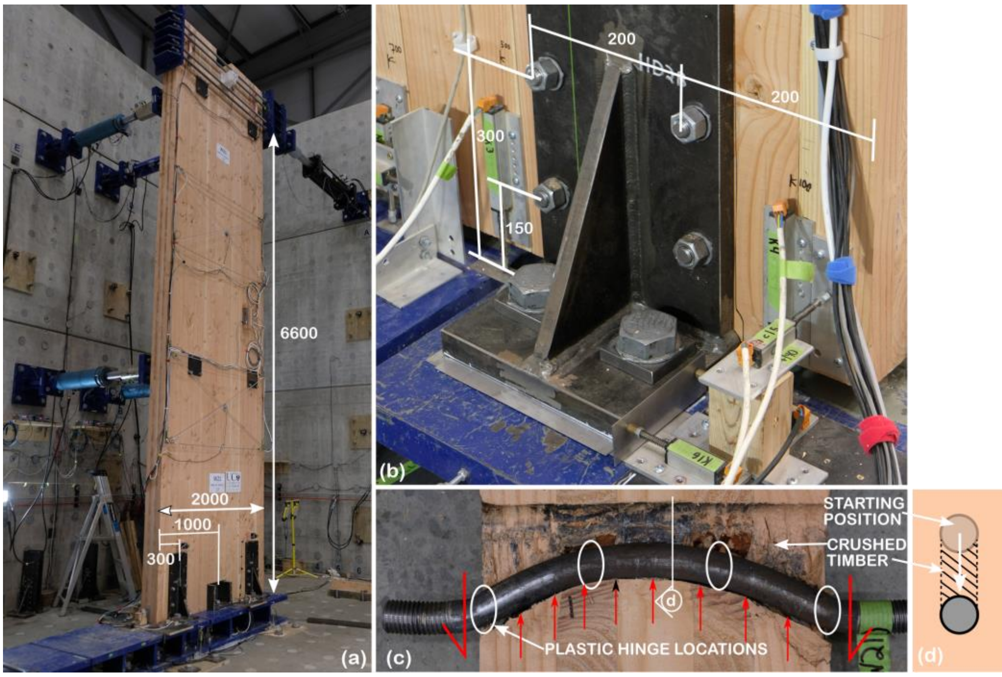

Figure 1: Cyclic testing of cantilever CLT wall with bolted hold-downs: (a) test setup, (b) hold-down, and (c) internal damage to connection.

The global hysteresis at the loading point is shown in Figure 2. The force-displacement plot shows the system behaviour is severely pinched and demonstrates a significant amount of ductile capacity. Additionally, the post-yield stiffness was significant and is due to an increasingly large rope effect (axial force in the bolts) creating friction between the CLT surface and steel side plates. Ultimate failure was not achieved due to a limited ram stroke of +/- 300mm in the test setup.

It should be noted that the bolted hold-downs exhibited significant post-yield stiffness and the peak strength occurred at approximately three times the nominal strength in the cantilever wall experiments. This is partly due to material overstrength, strain hardening, and inherent conservatism in the European yield model (EYM) (Johansen 1949; Blaß and Sandhaas 2017) which is used to determine the strength of the bolts. For bolts, the EYM includes a "rope effect" term in the strength calculation which accounts for the bolt's axial force and corresponding friction force at the contacting surfaces. Eurocode 5 (European committee for Standardization (CEN) 2003) places a limit on the amount of "rope effect" that is allowed in calculation of the bolt strength. However, there is no theoretical basis for this limit and therefore, it may reasonable to remove the cap on the rope effect term when determining the ultimate strength of a bolt in a CLT wall for ULS earthquake design. However, this study used the cap on the rope effect term for the design in the following section.

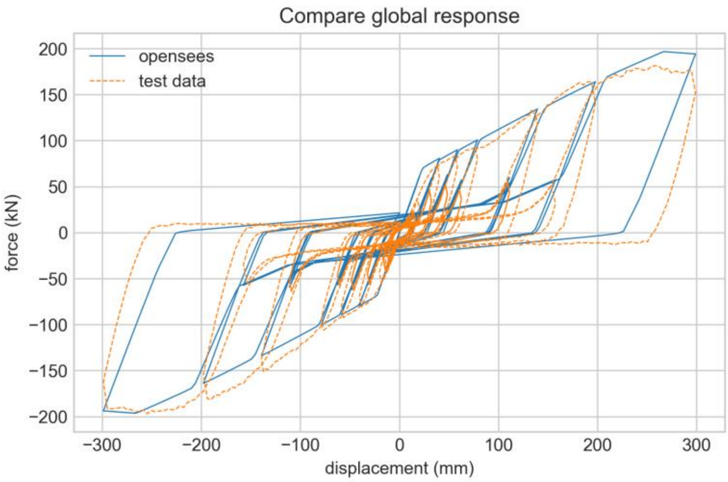

Figure 2: Global hysteretic behaviour of CLT wall experiment and comparison to Opensees model.

The pinched hysteretic behaviour occurs when the hold-down bolts are cyclically loaded as the bolts bend and locally crush the timber creating a void. Then as the hold-down is unloaded (or reloaded after a reverse cycle), the bolt must travel through the void before bearing on timber again. This pinched behaviour is typical for cyclically loaded timber systems (Foliente 1995) with steel dowel type or nail fasteners such as plywood shear walls (Stewart 1987), CLT walls with nailed connections (Dong et al. 2020; Gavric, et al. 2015), and dowelled hold-downs (Ottenhaus et al. 2018). Pinched systems have significantly less hysteretic energy dissipation and therefore less total damping when compared to typical steel or reinforced concrete systems. However, repeated cyclic loading also reduces the stiffness of the system and effectively increases the modal periods of the structure which can reduce the spectral acceleration for the effective first mode period.

In recognition of the pinched hysteretic behaviour, a deflection amplification factor, $k_{dt}=\mu^{0.5}$ is proposed in the *NZ Wood Design Guide: Seismic Design* (WPMA 2020) (including factor derivation) and the *BRANZ Multi-storey Light Timber-Framed Buildings Design Guide* (BRANZ 2019), to amplify the expected deflections of the system calculated from a linear elastic model as part of the equivalent static method. It was also proposed in a previous edition of the draft for *AS/NZS 1720.1 Timber Structures Standard* (Australia/New Zealand Standard 2018).

Stewart (Stewart 1987) conducted time history analyses on single degree-of-freedom nonlinear oscillators representing plywood shear walls with first mode periods from 0.1 to 2 seconds. The pinched hysteretic model was calibrated to several cyclic experiments and sensitivity analysis was completed on a variety of parameters in the model. Elastic-perfectly plastic oscillators with the same yield strength were also subjected to the same ground motions. Differences in ductility demand between the pinching and elastoplastic oscillators were "-generally less than 20%" and the average ductility ratio ($\mu_{\Delta,pinch}/\mu_{\Delta,ep}$) was 0.96, although the pinched oscillator had less than 50% of the hysteretic damping. Stewart attributed the similar response to the elongated period (reduced effective stiffness). However, the post-yield stiffness and therefore greater peak strength of the pinched oscillator is a significant factor to consider. The results of this study suggest that for a pinched system with a comparable yield strength to an elastoplastic system, no k_{dt} factor is. Also, this study was limited to only 4 ground motion records and may not be representative for a greater sample size.

Additional insights can be gained into pinched system behaviour by examining the design factors used for steel tension-only bracing systems (Filiatrault and Tremblay 1998; Bruneau et al. 2011). The AISC steel design standard AISC 341-16 (AISC 2016) categorizes tension-only bracing with ordinary concentrically braced frames which qualify for seismic design factors in the American standard ASCE 7-16 of R=3.25,

Ω_o=2, C_d=3.25. The corresponding force reduction factor in the NZ seismic code 1170.5 is k_μ=R/Ω_o=1.6. It can be observed that for the tension-only bracing system, with severely pinched hysteresis, the deflection amplification factor is equal to the force reduction factor (C_d=R), which also suggests no k_{dt} factor is needed. However, two cantilever CLT walls with bolted hold-downs have two notable differences when compared to tension-only bracing: (1) greater post-yield stiffness (relative to initial stiffness) and (2) greater hysteretic damping in pinched region (as compared to shake table test results (Tremblay and Filiatrault 1996)).

3 PROTOTYPE BUILDING

To investigate the adequacy of a force-based design approach, a prototype building was assumed (Figure 3) in the Christchurch area with an interstorey height of 3.5m and designed using the equivalent static method in NZS 1170.5 (New Zealand Standard 2016). Typical seismic weights of 3kPa and 2kPa were assigned to the floor and roof, respectively. The horizontal earthquake actions for design were calculated with a ductility factor of μ=3, structural performance factor of S_p=0.7, importance class IL2, a near fault factor of 1, and assumed soil class D. A flexible diaphragm was assumed and an equal tributary seismic mass was assigned to each of the 20 CLT walls in the building.

The equivalent static method was used to determine the base moment of a typical wall and the tension force demand of the hold-down was calculated using the method proposed by Reynolds et al. (Reynolds et al. 2017). The strength of a single bolt was determined as 77kN using the European yield model in Eurocode 5 (European committee for Standardization (CEN) 2003) with specified properties of f_y=300MPa and f_h=22.6MPa. Similar to the experiment presented previously, the bolts are made from a mild steel (f_y=300MPa) round bar by threading the ends. This was done to take advantage of the greater ductility of mild steel (relative to common grade 8.8 bolts) and encourage a ductile failure mechanism by reducing the yield moment of the bolts.

The first mode periods (T_1) were calculated using the Rayleigh Method (Cl. 4.1.2.1 of NZS 1170.5) and are summarized with other key parameters from the equivalent static method in Table 1. The hold-down specified for the 3-storey building is the same as the connection shown in Figure 1b.

Table 1: Summary of parameters from force-based design of the prototype building.

Storeys	T_1 (sec)	C_d	W (kN)	V_{base} (kN)	CLT Wall and Hold-down
3	0.72	0.17	7680	1310	2.4m-wide, 5-ply (45/35/45/35/45) 4-⌀20mm bolts
6	1.1	0.13	16300	2120	3.2m-wide, 7-ply (45/35/45/35/45/35/45) 9-⌀20mm bolts

4 NUMERICAL MODEL

Models were created in Opensees, as shown in Figure 3, to capture the anticipated nonlinear behaviour of a single cantilever CLT wall for the two buildings. The model configuration and nonlinear elements were chosen based on a similar numerical model calibrated to the test data of the CLT wall experiments described previously (see model overlay in Figure 2Figure 1). Linear elastic beam-column elements were used for the

CLT walls and seismic mass were lumped at each floor level. The seismic weight was also applied as vertical gravity loads to the wall nodes and a leaning column element used to account for P-delta effects. The distribution of the tributary weight between the wall and the leaning column was conservatively assumed (10% on wall nodes, 90% on leaning column nodes) because a greater amount of vertical gravity loads on the wall helps to reduce the hold-down uplift demands.

A simple linear elastic spring at the base accounted for the sliding behaviour. The moment-rotation behaviour of the wall base was captured with a zero-length fiber element. Contact springs (compression only) were placed at a spacing of 100mm along the base of the wall and used the Concrete01 element to capture crushing behaviour at the wall ends when the local stress exceeded 30MPa. Hold down elements at each end of the wall used the modified Ibarra Medina Krawinkler (IMK) with pinched hysteretic response (Lignos 2008) and were calibrated to the experiments with 4-⌀20mm bolts. The 6-storey building model used a similar hold-down element but the force values were scaled by a factor of 9/4 to account for the greater strength of 9 bolts versus 4. Tangent stiffness-proportional Rayleigh damping was used to account for non-structural damping in the building and was conservatively assumed as 3% critical damping at the 1st and 2nd mode periods. This value is in the middle of the range of 1-5% suggested by Foliente (Foliente 1995) for the linear viscous damping ratio in most wood systems.

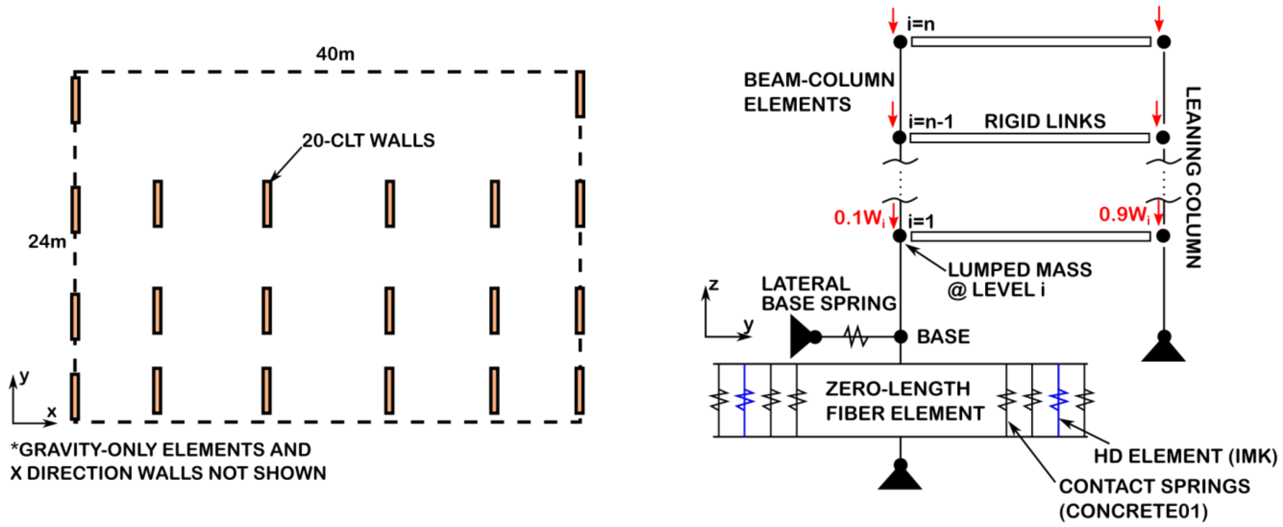

Figure 3: Building plan of prototype building for wall design (left) and schematic of Opensees model (right).

This study is limited to the nonlinear behaviour of the cantilever CLT walls and the model does not include the diaphragm, base shear connection, and other "gravity only" elements. Also, the 6-storey cantilever walls would require a capacity-protected splice at level three due to manufacturing/transportation constraints but this connection was not explicitly modelled.

5 NON-LINEAR TIME HISTORY ANALYSES

The walls designed using the equivalent static method were subjected to nonlinear time history analyses to determine whether the assumed ductility factor of three was appropriate.

5.1 Ground Motions

Twelve ground motions were selected from the PEER NGA-West 2 database (PEER n.d.) and scaled to the target spectrum which was determined according to NZS 1170.5 Section 5.5. Ground motions were scaled over a period range of $0.4T_1$ to $1.5T_1$. The acceleration spectra for the 6-storey building are shown in Figure 4. Two ground motions were selected from six shallow crustal earthquake events: Christchurch 2011, Darfield 2010, Chuetsu-oki 2007, El Mayor-Cucapah 2010, Loma Prieta 1989, and Northridge 1994. The ground motions were selected to be within the following parameters: v_{s30} between 150-300m/sec, distance to fault 15-30km, duration 0-30sec, and magnitude 6.5-7.5.

The two ground motion records from the Darfield 2010 earthquake, recorded at Christchurch Hospital and Christ's College in the city's central business district, show a unique peak in the 2-3 second range.

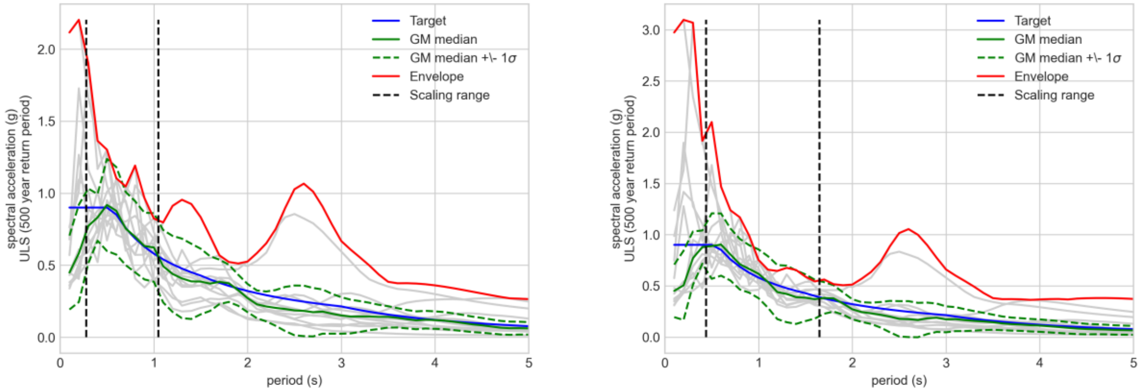

Figure 4: Target spectral acceleration and scaled ground motion spectra for the 3-storey (left) and 6-storey models (right).

5.2 Results and Discussion

The peak transitory interstorey drifts and ductility demands observed are summarized in Table 2. The ductility demands were calculated based on the peak transient roof displacement divided by the yield displacement from pushover analyses shown in Figure 5 ($\mu=\Delta_{roof}/\Delta_{roof,y}$). The mean ductility demand for the 3-storey building was 80% greater than the assumed ductility factor of $\mu=3$ used for the force-based design. However, the 6-storey building had a mean ductility demand of 3.4 which is reasonably close to the FBD value of 3.

Table 2: Summary of drift and ductility demands from nonlinear time history analyses.

	Interstorey Drift			**Ductility Demand**		
	Mean	Std Dev.	Max	Mean	Std Dev.	Max
3-storey	1.56%	0.41%	2.22%	5.4	1.5	7.8
6-storey	1.44%	0.39%	2.62%	3.4	1.1	6.6

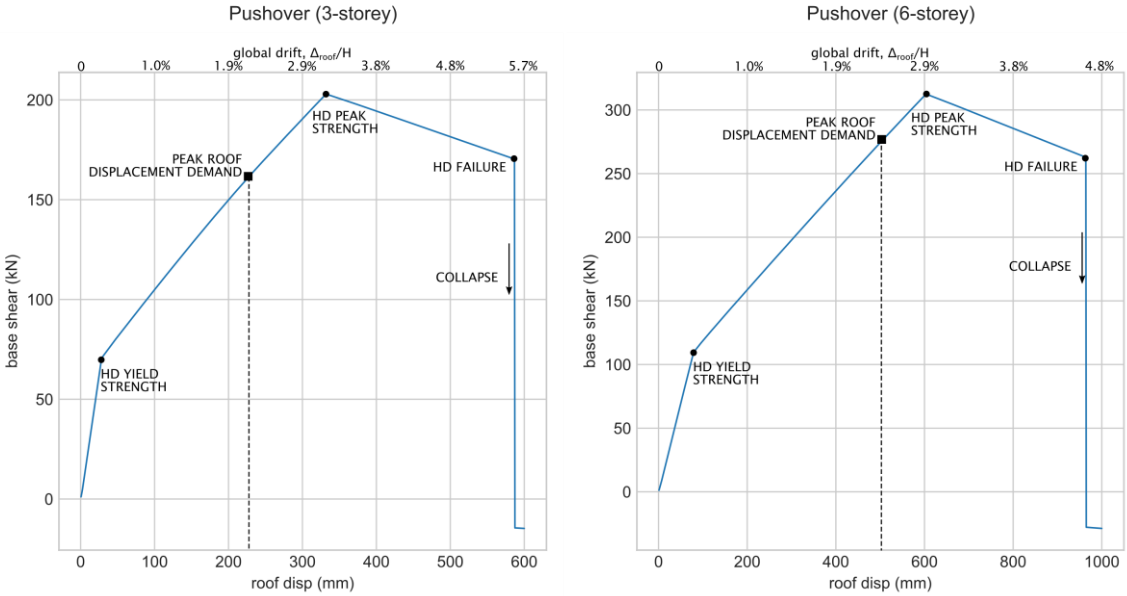

Figure 5: Pushover analysis of 3-storey (left) and 6-storey (right) wall models.

The displacement profiles for the numerical models (Δ_{nlth}) are shown by grey lines in Figure 6 at the time when their peak interstorey drift occurred for each ground motion. It is apparent that the lateral displacement is dominated by the base rotation of the wall in the 3-storey building but the wall deformation becomes more significant in the 6-storey structure.

The NZ seismic design code requires interstorey drifts found from the equivalent static method to be amplified by the ductility factor (μ), to account for inelastic deformation, and a factor k_{dm}, to compensate for the possible underestimation of drifts (Cl. 7.3 of NZS 1170.5). These profiles are shown by the red lines in Figure 6. This profile underestimated the mean response in the 3-storey model but slightly overpredicted the mean response of the 6-storey model. The different severity of the building responses is in part due to the ground motion scaling which was completed at different period ranges for each model.

An additional profile, using the k_{dt} factor to further amplify the lateral displacements, is also shown by the blue lines in Figure 6. While this profile provided a reasonable estimate of the upper bound displacement in the 3-storey model, it drastically overpredicted displacements for the 6-storey model by an average of 91%. The overprediction is in part because the inelastic behaviour of the wall occurs by rotation at its base level but the amplification factors were applied uniformly to the elastic displacement profile ($k_{dm}k_{dt}\mu\Delta_{ESM}$). Therefore, following a more rational approach, an additional profile was calculated and plotted (blue dashed lines) to only amplify the base rotation for inelastic action ($k_{dm}k_{dt}\mu\theta_b+\Delta_{ESM}$). The profile was closer to the mean response in both structures but still drastically over predicts the mean response for the 6-storey building. This may be due to greater period elongation occurring in the 6-storey model which typically reduces the effective spectral acceleration values (excluding the Darfield 2010 motions). Therefore, it appears that the k_{dt} factor is necessary to account for the reduced amount hysteretic damping of the pinched seismic behaviour but it may require some modification for longer period buildings (i.e. a reduced k_{dt} value or elimination for the 6-storey structure but no modification for the 3-storey structure).

One outlier response was observed in the results for the 6-storey model, as seen in Figure 6. This response occurred when the model was subjected to one of the Darfield 2010 ground motions (Christchurch Hospital) which created the peak at the 2-3 second range of the acceleration spectra envelope in Figure 4. The fundamental period of the 6-storey building model was 1.1 seconds but when damage occurred the period elongated due to a reduction in stiffness which would increase the effective spectral acceleration.

Additionally, the pinched behaviour reduced the hysteretic damping in the system resulting in the significantly greater displacement than the other records.

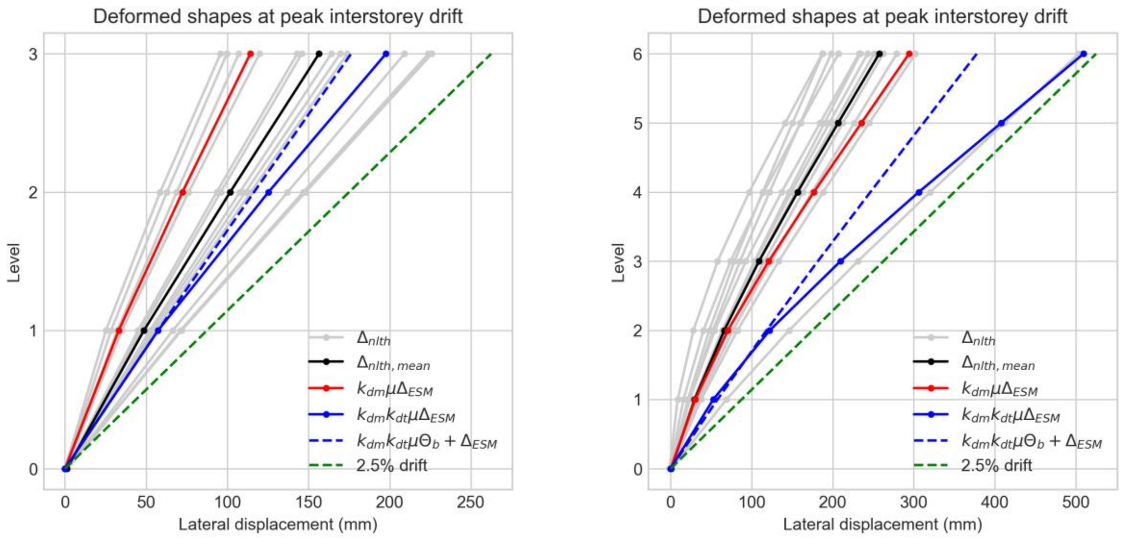

Figure 6: Lateral displacement profiles at peak interstorey drift for the 3-storey (left) and 6-storey models (right).

The behaviour of the walls was acceptable as none of the time history analyses exceeded the ULS limit of 2.5% (Cl. 7.5.1 of NZS 1170.5) and the wall behaviour did not exceed its peak strength. However, the probability of collapse is unknown without completing an incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) or the similar process described in the FEMA P695 Standard (Applied Technology Council 2009) which was out of the scope of this study.

6 CONCLUSIONS

This study included the review of the seismic behaviour of cantilever CLT walls with bolted hold-downs. Walls were designed using the equivalent static method from NZS 1170.5 with an assumed ductility factor of three for a prototype building of 3 and 6-storeys. The cantilever walls were then subjected to nonlinear time history analyses using 12 spectrum-compatible ground motions. Based on the results of these analyses, the following conclusions are made:

1. A ductility factor of three appears appropriate for the force-based design of cantilever CLT walls with bolted hold-downs for 3 to 6-storey buildings.
2. The use of k_{dt} to amplify lateral displacements was found to be necessary to amplify the peak displacements in the 3-storey building but not in the 6-storey structure. The factor slightly overpredicts the mean displacements of the 3-storey building by an average of 22% but significantly overpredicted the response of the 6-storey structure by an average of 91%.
3. The k_{dt} factor may require a reduction in magnitude for taller buildings with longer periods.
4. The Darfield 2010 ground motion highlights the potential for severe damage when a long period spectral acceleration peak is present for structural systems with pinched behaviour due to period elongation and reduction in hysteretic damping.

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