Effect of uncertainties in collapse assessment of coupled CLT walls with energy dissipators as couplers and resilient hold-downs.

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ABSTRACT

Rocking coupled CLT walls with energy dissipaters as couplers provide increased stiffness and damping, enabling the design realisation of taller CLT structures. Researchers have proposed design methods and conducted sub-assembly tests for this lateral load-resisting system (LLRS). However, implementing the LLRS remains a challenge for design engineers owing to performance when subjected to uncertainties related to; performance in the absence of full-scale testing, modelling, and design methods. Moreover, previous research on these types of walls as LLRS was focused on walls with post-tensioned (PT) cables. The LLRS consisting of coupled walls with self-centering hold-downs and energy dissipaters as couplers has been sparsely studied, especially for the collapse assessment with uncertainties. This paper presents the impact of appropriate uncertainties related to record-to-record variation, design methods, test data, and numerical modelling on the robustness of 6,8 and 10-storey archetype buildings with coupled CLT walls. The LLRS is evaluated using incremental dynamic analysis for MCE and beyond MCE-level earthquakes. The probability of a collapse was limited to 10% with appropriate consideration of uncertainties. Furthermore, higher mode effects on the performance of the system against collapse are discussed.

1 INTRODUCTION

Mass timber buildings with Cross-laminated Timber (CLT) walls as lateral load resisting system (LLRS) have attracted interest among design engineers and architects looking for sustainable and green design options. A viable LLRS with CLT walls has a predominantly rocking behaviour [1], even with damageable and pinching-prone conventional connections. Moreover, recent earthquakes have taught us that even the structures which performed as designed may need to be dismantled [2]. Thus, instigating a preference for low-damage design (LDD) systems among engineers. Contrary to this, designing structures with mass timber could be challenging due to its novelty [3] and especially developing connections for a low ductile system loaded with higher New Zealand seismic hazards [4].

Previous studies on low-damage LLRS with CLT walls [5], [6], [7], [1], [8], [9], [10], [11] were either focused on walls with Post-tensioned (PT) cables or were restricted to single wall systems with resilient dampers.

Rocking CLT walls with PT cables requires CLT to be machined for PT cables to pass through and may require monitoring of PT force (long-term creep in timber) which may reduce the system's competitiveness [8]. Moreover, as the high force in PT cables governs the response (overall damping gets reduced), the structure could experience very high undamped accelerations [12]. Similar high accelerations have been reported in the seven-storey shake table test of the CLT building [13].

Resilient slip friction joints (RSFJs) [14] as shear wall hold-down form an LDD system that offers self-centring and damping without the post-tensioned (PT) cables. However, CLT walls with relatively low inherent stiffness may require longer walls or coupled walls to realise tall structures. Further, increasing the wall length has an adverse effect on the ductility of the LLRS [1]. An increased aspect ratio (height of wall/ length of the wall) increases the displacement demand in the hold-downs, thus limiting the ductility of the LLRS. The optimum aspect ratio for CLT walls with conventional hold-down was observed between 3 to 4 [15]. Coupled CLT walls with energy dissipaters as couplers can provide higher stiffness and damping required for taller structures whilst maintaining a lower aspect ratio.

2 COUPLED CLT WALLS

In coupled wall system, the moment demand in the walls due to lateral force is converted into axial forces through vertical connections along the height of the wall. Traditionally, these connections are moment connections with a coupling beam between the walls. These beams act as the ductile link and undergo inelastic deformations under lateral loads providing ductility and energy dissipation [16].

Another way of coupling rocking timber walls is by reducing the gap between the rocking walls and attaching a shear damper between them [17] (U-shape flexural plate (UFP) [18] or equivalent system) or simply nailed plywood sheets [19] (refer to Figure 1). This coupling effect increases the system ductility and energy dissipation (through connections between the panels).

In-plane performance of coupled LVL (Laminated Veneer Lumber) walls was studied by Sarti et al. [20] and Iqbal et al. [19]. These LLRSs consisted of PT (Post-tensioned) cables and UFPs as energy dissipation devices riveted to the walls. The long-term creep and relaxation of timber and PT cables may cause a reduction of the PT force. This creep may require a higher initial PT force and constant monitoring/maintenance of the LLRS for the PT forces.

Moreover, the horizontal component of PT force in tall rocking walls could be more demanding for the total PT force applied on the LLRS [21]. Further, this high PT cable force will form the major component of the LLRS, thus reducing the force contribution of the dampers and thus reducing the damping of the LLRS. Therefore, as observed by Ceccotti et al. [13] in conventional structures, the issue of high floor acceleration remains unaddressed.

Brown et al. [9] investigated PT-coupled CLT walls with mixed-angle self-tapping screws. Further, Ganey et al. [22] also observed that increased PT forces might result in CLT damage at lower drifts. Therefore, the PT force should be carefully tuned. It should be large enough to take care of serviceability limit states and creep of timber while small enough not to damage CLT walls. Ganey et al. also observed pinching in these systems for more significant drifts.

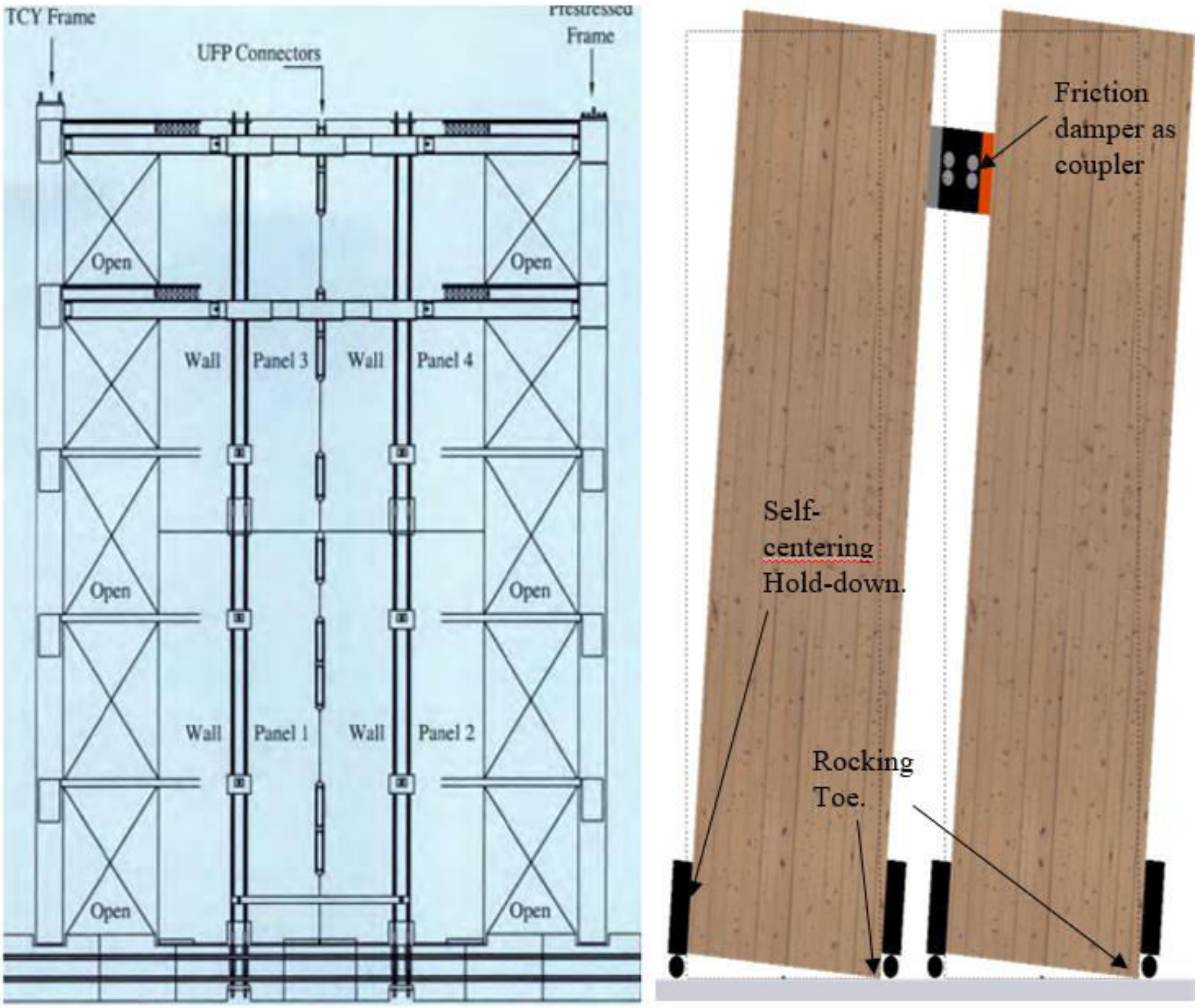

Figure 1 Coupled wall panels with direct shear link connection [17] (left), Coupled-wall with FD as coupler and self-centring hold-down (Right).

Dires and Tannert [23] experimentally investigated coupled CLT shear walls with internal perforated plate geometries as vertical connections and hold-downs at component and sub-assembly levels (full-scale shear walls). The LLRS achieved a drift ratio of 2.2%; however, full-scale monotonic tests indicated residual drifts without a self-centering force.

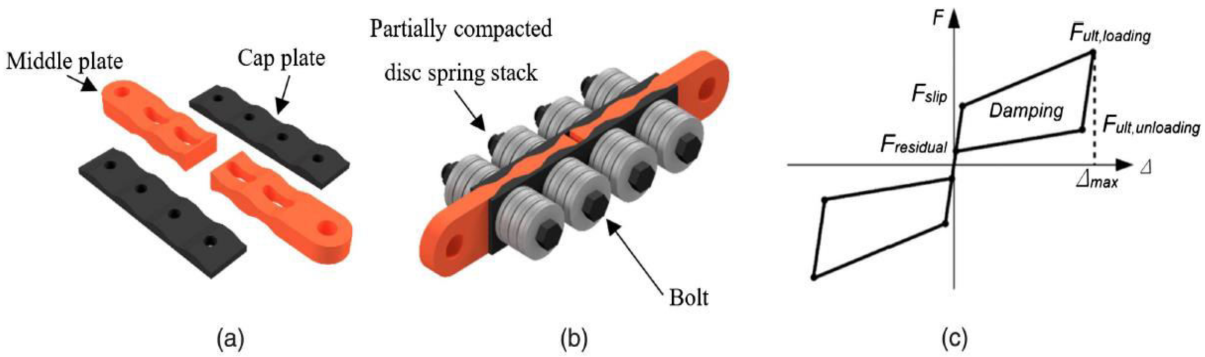

Figure 2 RSFJ a) Grooved sliding plates; b) assembly; and c) joint hysteresis [24]

Hashemi et al. [25] proposed a concept of coupled CLT walls with boundary gravity columns and resilient slip friction joints as Hold-downs. This study reported complete self-centering, high ductility and lower floor accelerations.

2.1 Resilient slip friction joints (RSFJ)

The resilient slip friction joint (RSFJ) is a self-centring friction damper invented in New Zealand by Zarnani and Quenneville [26]. The hysteresis is flag-shaped, with large areas (up to 20% hysteretic damping) between the loops providing self-centering and damping (refer to Figure 2). The RSFJ comprises sliding surfaces at an angle with groves accommodating bolts clamped with disc springs. The relative movement of the inclined sliding surface pushes the cap plate move outwards, elastically compressing the disc springs, thus creating a restoring force that drives the system back to its original location (refer to Figure 2). The energy is dissipated through friction damping between the middle and cap plate [27]. The reserve capacity beyond design (secondary fuse stage) is provided through the yielding of bolts. This secondary fuse provides 1.5 to 2.2 times the design displacement capacity. The system is tested and analytically established to remain self-centring even in the secondary fuse stage [28], thus eliminating any residual drifts in the LLRS.

In summary, an LLRS consisting of coupled CLT walls with energy dissipaters as couplers and self-centering hold-downs could be an efficient solution for taller mass timber structures (refer to Figure 1- right). This system will have more ductility without damage and considerably high damping without pinching. Moreover, residual drifts would be eliminated. However, the LLRS consisting of coupled walls with energy dissipaters as couplers and resilient hold-downs is a comparatively newer concept, and the effect of uncertainties related to the test data, design method, numerical modelling, and record-to-record variation needs to be investigated. This study summarises the various uncertainties for the considered LLRS based on available test data, literature and FEMA-695 procedure and further evaluates the robustness of the system performance (collapse prevention) for a conservative range of uncertainties.

3 METHODOLOGY

For conventional systems, seismic behaviour factors such as: structural ductility factor μ, inelastic spectrum scaling factor kμ, structural performance factor, Sp as in the New Zealand standards [29], and response modification coefficient R, overstrength factor Ω, and deflection amplification factor Cd, as in the American standards [30], give design engineers confidence to design an LLRS. These seismic behaviour factors inherently account for the uncertainties associated with the design process, test data, modelling and earthquake records.

These seismic behaviour factors are derived either by extensive analysis and tests [31] or through decades of experience with real-life earthquakes on the performance of the LLRS. FEMA P695 [31] lays down a robust procedure to derive these behaviour factors. In the absence of any methodology to evaluate uncertainties for newer systems in New Zealand standards, FEMA P695 [31] procedure is used to assess the system's robustness against the associated uncertainties.

The flowchart in Figure 3 briefs the steps involved in the study, which are described as follows:

3.1 System information

As part of gathering system information on the considered LLRS, the uncertainties of the design process for the components, sub-assembly and complete assembly is evaluated. Further, the test data's uncertainties are assessed both for comprehensiveness and variations.

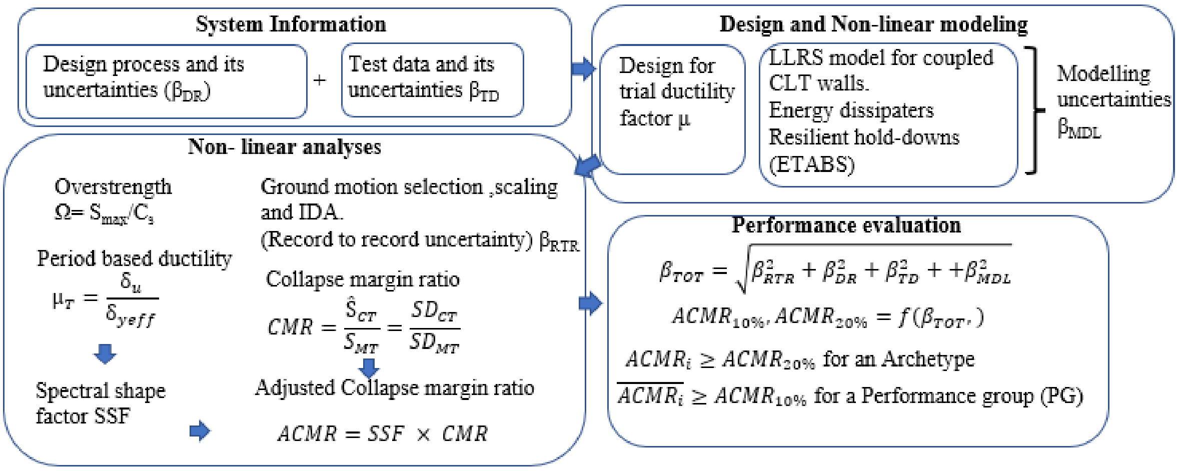

Figure 3 Evaluation of uncertainties as per FEMA P695

3.1.1 Design process and uncertainties

The coupled CLT wall system consists of self-centering hold-downs, energy dissipaters (friction dampers in this case), and CLT walls.

The design methods of CLT walls and their connections have been tested, evaluated and published in international standards [32], [33], and national literature [9], [24] [34]. These standards cover the uncertainties related to the design of CLT as the LLRS component. Further, the CLT and its connections are capacity designed for the considered LLRS as the non-linearity is restricted to the hold-downs and couplers. Moreover, the performance of LLRS with CLT walls is found to be governed by the hold-downs, and the wall itself behaves rigidly [1], [9], [13], [15], [23], [25]. The hold-downs and energy dissipating coupler considered are also designed to standards and are 100% tested. Therefore, the design process of the LLRS provides superior confidence and comprehensiveness.

In addition to the equivalent lateral force method to determine the capacities, DBD (displacement-based design) method with performance verification using ADRS (Acceleration displacement response spectrum) is specified in the design process. The DBD method along with the performance verification using ADRS is the most extensively used design method (other than the non-linear time history procedure) for designing LLRS with energy dissipaters [35], [36], [37]. This method has been established both analytically and through tests [38], [39], [40], [41].

Further addition of design considerations, such as displacement compatibility of the diaphragm connection (unrestrained rocking movement), drift limit requirements for the gravity system, factor for higher mode effects in the design of shear load path, impact factor for rocking toes, and conforming damper displacement capacity to 1.3 times MCE drifts adds to the comprehensiveness of the process [1] [3]. Thus, this design method provides high confidence and reliability [31].

The design process provides reasonable safeguards against unanticipated failure modes. However, owing to the limitation of the linear and non-linear static methods of design, the process can be rated as high to medium in completeness and robustness. Thus, following the quality rating design requirements of FEMA P695, a design process uncertainty (β_{DR}) value of 0.2 could be adopted for the study. However, this study intends to do a conservative evaluation of the LLRS. Thus, the design process uncertainty is chosen as Medium to fair with a value of β_{DR} as 0.35 (refer to Table 3-1 of FEMA-P695 [31]) for the investigation purpose.

3.1.2 Test data and uncertainties

Component testing of CLT walls as a component of LLRS has been tested and established [42], [43], [44], [45], [1]. As discussed earlier, the walls were ascertained as rigid elements, while ductility was restricted to the connections. Moreover, internationally CLT is manufactured as per published standards minimising the uncertainties in the LLRS's component.

Further, component tests and full-scale sub-assembly tests of RSFJ's (Self-centering hold downs) have established the hysteretic variation within 5% [24], [27], [28], [46], [47]. The results of dynamic tests also confirmed a minuscule variation in performance (~5%).

Energy dissipaters chosen here are friction dampers confirming ASCE-07 and ASCE-41 [30], [36]. The standard restricts the variation to 15%. This variation accounts for the variation due to ageing, prototype testing, and manufacturing. However proprietary friction dampers can have variations limited to 5% of the designed displacements and forces.

Additionally, a two-storey full-scale shake table test by Blomgren et al. [8] and Pie et al. [48] establishes the diaphragm connection with rocking walls and performance of the mass timber gravity frame with or without PT cables (with replaceable components). These tests also confirmed the compatibility of the CLT wall with the gravity frame. Thus, reducing the uncertainties related to the test data for the use of the rocking coupled CLT wall as LLRS.

The performance of coupled mass timber walls, including CLT walls, has been tested and established by various researchers [8] [19], [18], [22], [48], [49], [50], [51]. These tests included PT cables or conventional hold-downs. The test data of coupled CLT walls with a self-centering system hold-down is not available yet and is planned by the authors.

The above discussed studies have produced high quality of test data with consideration to all the key parameters of the system. As a result, the test data instils high confidence in the system's parameters. However, in the absence of full-scale tests of coupled CLT walls with self-centring hold-downs for taller CLT structures, the completeness rating may be considered high to medium. Thus, the test data uncertainty (β_{TD}) value of 0.1 to 0.2 could be easily considered for the study. Nevertheless, a high β_{TD} value of 0.35 is selected to conservatively verify the robustness of the system.

3.2 Design and non-linear modelling

A mixed-use building plan chosen for the study was analysed with 6,8, and 10 storeys (refer to Table 1). The wall and damper sizing were based on forces from the equivalent static method assuming ductility (μ) of 3. The damper's force capacity was calculated from static equilibrium with conditions that the walls are constrained rotationally. Further the energy dissipator (coupler) force is small enough not to lift the coupled wall from the base. This required the slip force of the coupler to be half the ultimate force of the hold-downs.

The displacement capacity of RSFJs was kept equal to 1.3 times the MCE demands derived from the ADRS curve, while the coupler displacement was designed to accommodate the secondary fuse displacement of the RSFJ. For the full design process, the reader is requested to refer literature by Agarwal et al. 2021 [3].

The CLT walls and its connections were capacity designed to suppress any un-simulated failure modes. The CLT walls were modelled using effective young's modulus and shear modulus as per the CLT handbook [33]. The numerical model was verified by matching displacement under static loads with the analytical equations. This CLT modelling technique used has already been established by researchers [45], [52]. Besides as discussed earlier, past studies suggest that the non-linearity in LLRS with rocking CLT walls is restricted to the coupler and hold-down [1], [8], [9], [13], [15], [23], [25].

Commercially available software ETABS [53] was used to model the LLRS.

Table 1 Building information of archetype buildings

Building dimensions	14.7x22.8 m	No. of LLRS grid	5 Nos with 2 coupled walls each
storey height 1st	4.5 m	Wall length (6/8/10 storey)	3.5/4.5/5.5 m
storey height 2nd	3.2 m	Wall thickness	315 mm (7 layers)
storey height 3-6 /8/10	3.2 m	Modal time-period (6/8/10 storey)	0.64 to 0.86 sec
storey weight 1st	1306 kN	Hazard factor Z	0.3
storey weight 2nd	1137 kN	Soil type	D
storey weight 3-6 /8/10	879 kN	Diaphragm	CLT
Wall aspect ratio	High (6)	Gravity loads category	Low

The RSFJs were modelled using a damper friction element in ETABS [53]. These devices can be modelled in any software capable of modelling a flag shape hysteresis. The model was verified by researchers [52], [54] [55] to be accurate within 5% of the analytical model and the test results. Further, the flag-shaped hysteretic behaviour of the RSFJ is verified to be repeatable without degradation enabling it to be modelled more accurately than other degrading and pinching type connections.

The friction damper couplers with rectangular hysteresis were modelled with isotropic multilinear plastic links, accurately depicting their hysteresis. These are a common type of energy dissipaters which have been in use for the last four decades [56], [57], [58]. The modelling process and hysteresis are well-established and available in most research and commercial software.

The non-simulated failure modes (in members modelled as elastic elements) were evaluated through checks on demand from the non-linear analysis.

In summary, numerical model which can accurately capture the full range of structural behaviour of the LLRS until collapse can be easily developed. Considering Table 5-3 of FEMA P695, a low modelling uncertainty (β_{MDL}) of 0.1 could be selected for the LLRS; however, a value of modelling uncertainty of 0.2 (β_{MDL}=0.2) is chosen to retain conservatism in the study.

3.3 Non-Linear analysis

The FEMA P695 methodology for the evaluation of uncertainties requires incremental dynamic analysis (IDA). The numerical model is evaluated against collapse with increasing spectral acceleration corresponding to its fundamental period. The spectral acceleration at which 50% of the ground motion leads to collapse ($\hat{S}_{ct}$) is determined. Further, the Collapse Margin Ratio (CMR) is defined as the ratio of $\hat{S}_{ct}$ to the spectral acceleration at the MCE level of hazard (S_{MT}).

This CMR is factored to the Adjusted Collapse Margin Ratio (ACMR) to normalise the overestimation of the non-linear response due to the scaling of very rare ground motion to high values through a factor called spectral shape factor (SSF). SSF, which is the function of period base ductility (μ_T), accounts for the fact that as the ductile structure softens, there is a more rapid drop of response in rare records than expected based on the standard spectrum. For this study, the period base ductility of the structure exceeds 3. However, a conservative value of SSF is chosen as 1.22 [31]. The period-based ductility (μ_T) is the ratio of yield displacement to the ultimate displacement causing collapse. This should not be misinterpreted with inelastic scaling factor (kμ) or design ductility μ.

A non-linear direct integration method was employed for the analysis. Two percent inherent damping was considered for this study. The pivots for the Rayleigh damping model were selected as the first mode period and the period of the mode exceeding 90% mass participation. This ensures that higher modes effecting collapse response are adequately represented while dissipating energy in very high frequencies to avoid numerical errors.

3.3.1 Ground motion selection and scaling

A set of 44 high-magnitude, far-field ground motions specified by FEMA-P695 was selected for this study. These ground motions were normalised by their peak ground velocity to remove the biases due to magnitude, source distance, source type and site condition while retaining frequency content and the record-specific characteristics governing the collapse mechanism. These records were initially scaled such that the median spectral acceleration of the records at the fundamental period of the archetype matches the MCE spectra. The scale factor for the three archetypes was in the range of 0.91 to 1.05, indicating a good fit. The uncertainty for the ground motion records (β_{RTR}) selected is pegged as 0.4 by FEMA P695. This uncertainty value is for the specified set of ground motion covering a broader range of hazard. This uncertainty values and the records may contain biases for the US seismic scenario. A record set with uncertainties is required to be developed for the NZ scenario for future uncertainty evaluation.

3.4 Performance evaluation

The total system uncertainty (β_{TOT}) is evaluated from various uncertainties using equation 1 as per FEMA P695.

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + +\beta_{MDL}^2} \qquad (1)$$

This β_{TOT} independently define the lognormal standard deviation λ_{TOT}. Where λ_{TOT} is the product of four variables λ_{RTR}, λ_{DR}, λ_{TD}, λ_{MDL}, defined as independent lognormally distributed variables with median values of unity, and lognormal standard deviation parameter as the corresponding uncertainties evaluated (β_{RTR}, β_{DR}, β_{TD}, β_{MDL}).

Generally lognormal collapse probability is controlled by $\hat{S}_{ct}$ and β_{RTR}. The $\hat{S}_{ct}$ corresponds to median collapse intensity while β_{RTR} reflects the dispersion in results due to record to record uncertainty. In this study the β_{RTR} is fixed to 0.4 considering FEMA P-695 value based on various studies and low impact on CMR values when combined with various other uncertainties.

Further, the collapse fragility is defined by the variable S_{CT}, calculated using equation 2.

$$S_{CT} = \hat{S}_{CT}\lambda_{TOT} \qquad (2)$$

The increase in uncertainty tends to flatten the collapse fragility curves and increase the corresponding collapse probability at MCE-level earthquakes, while the 50% collapse probability remains pivoted at $\hat{S}_{ct}$..

For the selected archetypes, the total system uncertainty is calculated as 0.667. The actual system uncertainty could range between 0.435 to 0.529. A full assembly test confirming the performance of coupled shear walls with self-centering hold-downs will provide superior confidence in the realisation of the discussed LLRS with reduced uncertainties.

Based on the above, ACMR values for the selected 6, 8 &10 storey buildings were calculated as 2.3, 3.5 and 3.6, respectively.

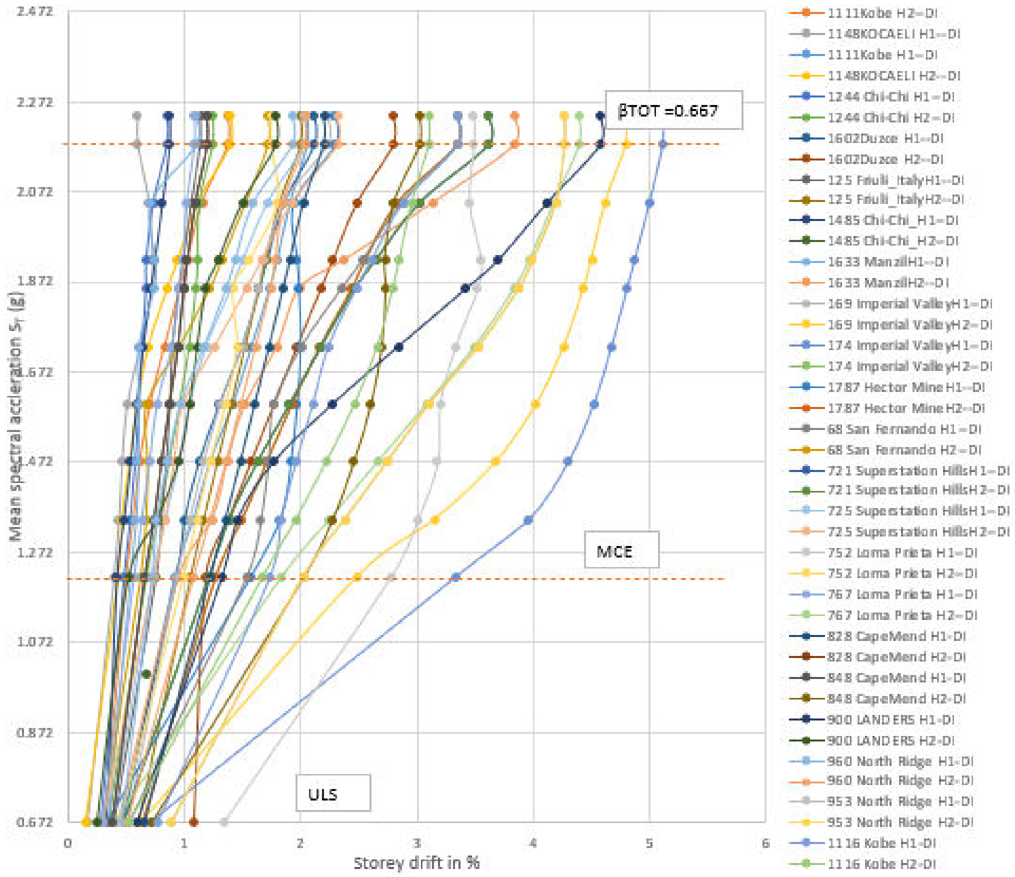

Figure 4 ACMR evaluation of 8-storey Archetype building - IDA.

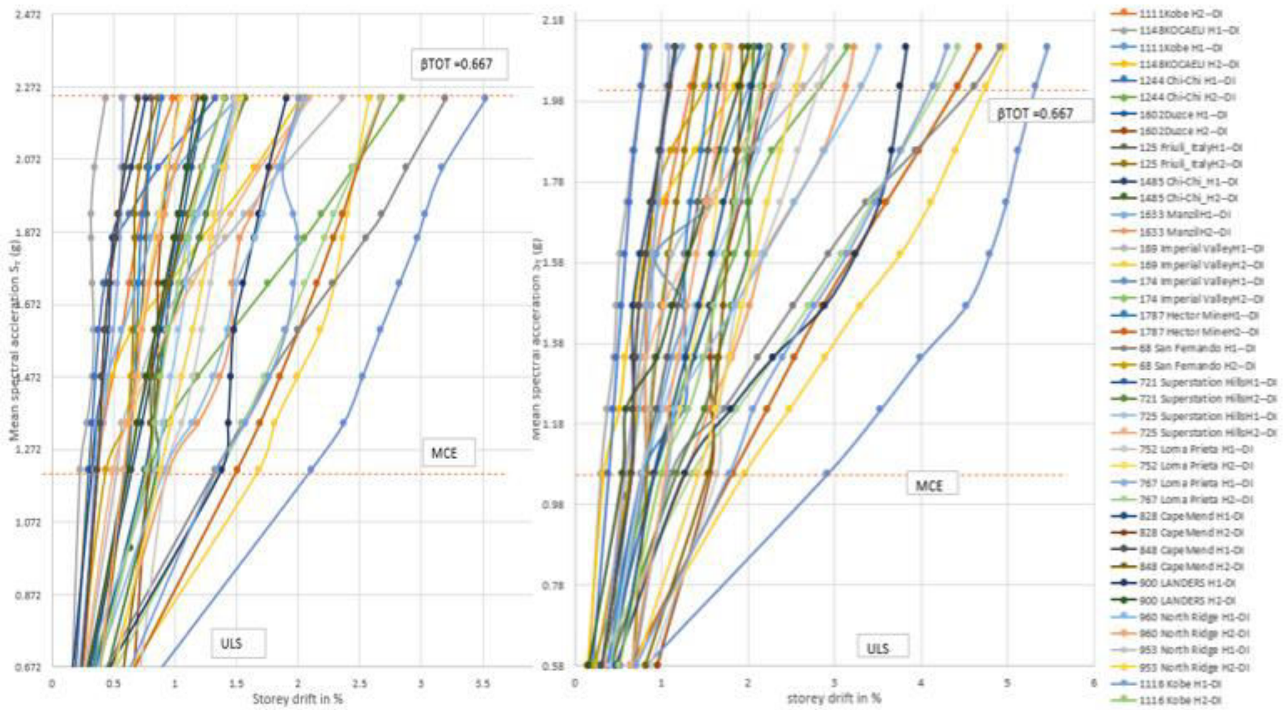

Figure 5 ACMR evaluation of 10-storey Archetype building – IDA(L) and 6-storey Archetype building(R)

4 RESULT AND DISCUSSION

The IDA analysis was carried out to the required ACMR values back calculated from the uncertainties. The collapse criteria limit of 5% drift was also included for gravitational system compatibility [48]. Please note that the results does not include complete IDA analysis from low spectral acceleration to complete collapse in each ground motion, but is limited to the range from spectral acceleration corresponding to MCE to the accelerations corresponding to required ACMR values. The linear portion (no- slipping) of the analysis and higher acceleration values is not included in the graphs. Moreover, the secondary stiffness of the considered LLRS even in the secondary fuse stage of the hold-downs is positive preventing sudden slope change in the IDA curves after yielding. This secondary stiffness remains relatively in the same range from the first slip to the final collapse of the structure. Thus, this truncation of IDA analysis and the unique secondary stiffness make the shape of the IDA considerably different than the conventional IDA's.

The analysis suggested very high structure robustness at acceptable ACMR values which corresponds to a conservative value of uncertainties. Nevertheless, less than 25 % of records indicated collapse at the acceptable ACMR values compared to the 50% limit per FEMA695.

The mean moment of the records at the scaled MCE intensity was less than the design base moment at ULS, confirming the little moment amplification due to higher mode effects. Though, there was a magnification of mean shear force. However, the larger inherent shear capacity of the CLT walls counterpoised failure due to this shear amplification. The numerical models accounted for the CLT wall's shear stiffness, but the shear keys were modelled as rigid toes. The LLRS needs to be investigated through appropriate modelling of shear key stiffness and shear springs along the height of the wall representing the construction joints. Another analogy for shear amplification could be that the displacement (thus damping) of the energy dissipaters (both coupler and hold-down) in the considered LLRS is more moment governed. This results in high shear inertia of the lower storeys (producing lower moments) undamped. However, this needs to be further investigated.

The DBD-ADRS design drifts at ULS and MCE levels were conservative to those obtained from the analysis at the corresponding levels. Nonetheless, there was about 10% base shear amplification at the MCE level records pertaining to higher mode shear amplification, which was included in the design considerations. Thus, the robustness of the design procedure using DBD-ADRS was verified. The Equivalent hysteresis damping (EVD) did not require any correction for the selected LLRS.

5 CONCLUSION

Three archetype buildings of 6,8 &10 storeys were assessed for uncertainties pertaining to test data, design process, modelling and earthquake record to record based on FEMA P695 methodology. The walls assessed had a high aspect ratio (in the range of 6) pertaining to the mid to high-rise range of archetype selection. The LLRS comprised of coupled CLT walls with self-centering hold-downs and energy dissipater as coupler was assessed with a conservative value of total system uncertainty as 0.667. The system indicated robustness against collapse with less than 10% probability of collapse at the MCE level of earthquakes.

The LLRS indicate the potential of designing with higher ductility even with conservative values of uncertainties. Favourable results could have been obtained if more realistic values of uncertainties, in line with available data, were selected. A comprehensive analysis, similar to the one performed in this study can be used to derive seismic behaviour factors for the LLRS.

The DBD-ADRS design method conservatively predicted the design drift without correction in the equivalent hysteretic damping. The base shear amplification due to higher mode effects needs further evaluation, which was counterbalanced by the larger inherent shear capacity of the CLT walls. However, no magnification of moments was observed for the LLRS due to higher mode effects.

6 ACKNOWLEDGEMENT

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