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Design and performance uncertainties for single and coupled Cross-laminated Timber walls with resilient dampers

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ABSTRACT

Rocking CLT walls with resilient dampers enables the realisation of more robust and taller mass timber structures. Recent research has laid out design methodologies and modelling guidelines for these lateral load-resisting systems (LLRS). However, most of the previous research on these types of LLRS is focused either on conventional hold-downs or walls with post-tensioned cables. Moreover, accounting for uncertainties in these methodologies, modelling techniques and performance due to the lack of full-scale test data poses a challenge for design engineers to adopt these LLRS. This paper presents the effect of uncertainties related to record-to-record variation, design methods, test data, and numerical modelling on the robustness of selected archetype CLT buildings with resilient self-centering hold-downs and energy dissipaters as couplers. The archetype buildings selected for this study consist of multi-storey residential, open-plan commercial, and mixed-usage buildings with coupled CLT walls. The archetypes selected are 6,8 and 10 stories high with wall-wall coupling and wall-column coupling with energy dissipaters. The LLRS is evaluated using incremental dynamic analysis following the FEMA-P695 methodology. Furthermore, key parameters affecting performance and design, such as higher mode effects, and the range of correction factors for equivalent viscous damping for the LLRS, are presented.

Keywords: CLT walls, coupled walls, energy dissipaters, damping, uncertainties, resilience.

INTRODUCTION

Mass timber buildings with Cross-laminated Timber (CLT) have emerged as a choice of construction material among design engineers and architects as a sustainable and green design option. CLT walls as lateral load resisting system (LLRS) has been researched since 2004 [1] and comprehensively in the last decade [2]- [3].

A practical LLRS with CLT walls has a predominantly rocking behaviour, even for platform-type constructions of up to 6 storeys high [4]. Further, the damage in CLT walls is restricted to connection / hold-downs, with CLT panels behaving rigidly [5]. Moreover, recent earthquakes have taught us that even the structures which performed as designed may need to be dismantled [6]. Consequently, the utilisation of an advanced Low Damage Rocking Wall LLRS is a more judicious option. Conversely, designing structures with mass timber could be challenging due to its novelty [7], particularly in developing connections for a low-ductile system that is subjected to heightened seismic hazards [8]. Earlier investigations on low-damage LLRS incorporating CLT walls as LLRS [9] [10] [11] [12] [13] [14] [15] have either concentrated on walls featuring Post-Tensioned (PT) cables or have been limited to single-wall systems incorporating resilient dampers.

Rocking CLT walls with PT cables may require costly machining and monitoring of PT force (due to long-term and high creep in timber), thereby reducing the system's competitiveness [13]. Furthermore, the high force in PT cables governs the structure's lateral response and reduces the damper's damping component, making it vulnerable to significant undamped accelerations [16]. Similar high accelerations have been reported in the conventional seven-storey CLT wall-building system subjected to shake table tests [5].

Resilient slip friction joints (RSFJs) [17] as shear wall hold-down form an LDD system offers self-centring and damping without the post-tensioned (PT) cables. Nevertheless, CLT walls exhibiting relatively low inherent stiffness may necessitate longer or coupled walls to construct taller structures. However, increasing the wall length has an adverse effect on the ductility of the LLRS [4]. The increased aspect ratio (height/length of the wall) of the LLRS results in higher displacement demand on the hold-downs, which restricts the system's ductility. The optimal aspect ratio for CLT walls with conventional hold-downs is 3 to 4 [3]. However, using coupled CLT walls with energy dissipaters as couplers can achieve higher stiffness and damping while maintaining a lower aspect ratio for taller structures.

COUPLED CLT WALL

Vertical connections in a coupled wall system convert moment demand into axial forces. Traditionally, a coupling beam with moment connections acts as ductile links to provide energy dissipation through inelastic deformation under lateral loads. [18].

Coupling rocking timber walls can be achieved by reducing the gap between them and attaching a shear damper, such as a U-shape flexural plate [19] or a friction damper (refer to Figure 1a). This enhances the ductility and energy dissipation through connections between the panels [20]. However, the wall lengths (sum of both) could be impractical for higher timber buildings for the corresponding CLT floor span. Therefore coupling with boundary columns could be more preferred (refer to Figure 1b).

Sarti et al. [21] and Iqbal et al. [22] studied the in-plane performance of coupled LVL walls with PT cables and UFPs as energy dissipation devices. However, long-term creep and relaxation of timber and PT cables can cause a reduction of the PT force, requiring constant monitoring/maintenance of the LLRS. Ganey et al. [23] found that excessive PT forces could damage CLT walls at lower drifts, requiring careful tuning of PT force to balance serviceability limit states and timber creep. Additionally, pinching was observed for larger drifts in these systems. Brown et al. [24] investigated PT-coupled CLT walls with mixed-angle self-tapping screws observing the loss of coupling effect and damage at high drifts.

Dires and Tannert [25] studied coupled CLT shear walls with internal perforated plate geometries (as hold-downs and couplers) and achieved a drift ratio of 2.2%, but full-scale tests showed residual drifts without self-centering force. Hashemi et al. [26] proposed coupled CLT walls with boundary gravity columns and Resilient slip friction joints (RSFJ) as hold-downs. Owing to the advantages of self-centering, high ductility, and lower floor accelerations of the system proposed by Hashemi et al., a similar system is investigated in this study.

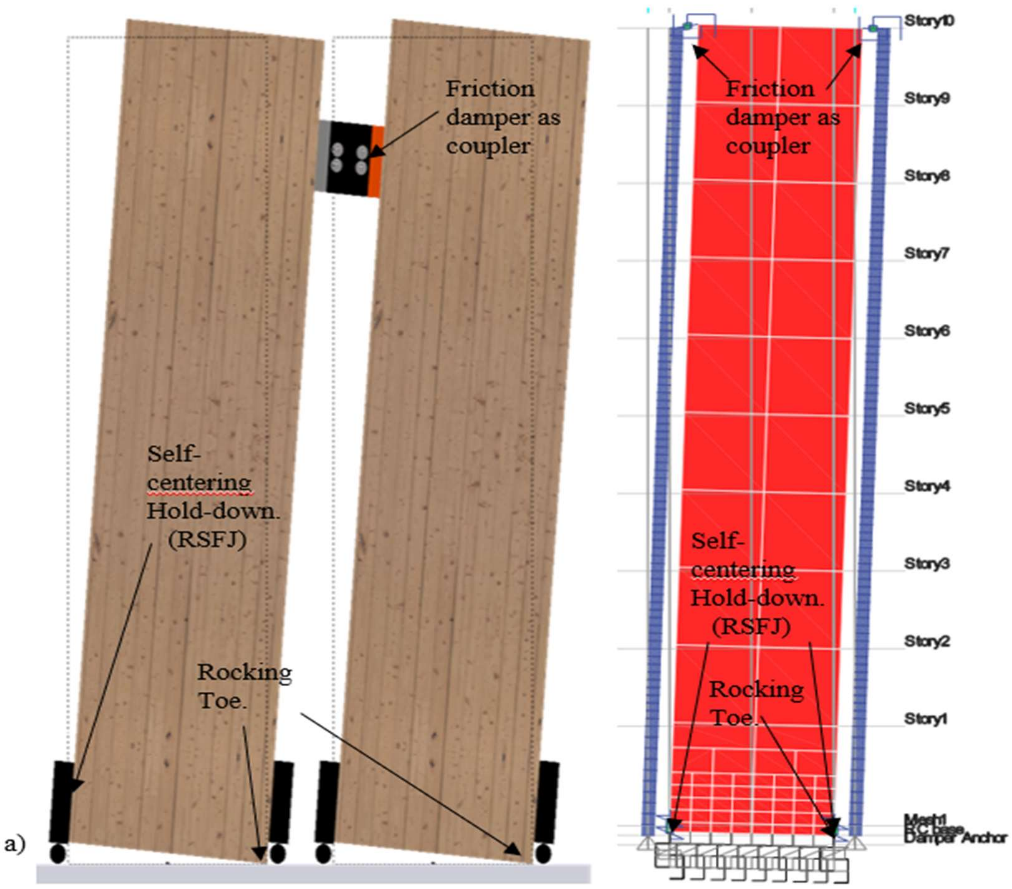

Figure 1 a) Coupled-wall with wall. b) Coupled wall with boundary columns

Resilient slip friction joints (RSFJ)

The resilient slip friction joint (RSFJ) is a self-centering friction damper with a flag-shaped hysteresis (refer to Figure 2c)., invented in New Zealand [27]. It has inclined sliding surfaces (with a special coating), and slots to accommodate bolts clamping the disc springs (refer to Figure 2b). During extension (or contraction), the elastic compression of disc springs creates a restoring force that drives the system back to its original location, dissipating energy through friction damping (refer to Figure 2a). The secondary fuse stage, provided by the yielding of bolts, provides 1.5 to 2.2 times the design displacement capacity. The system remains self-centering even in the secondary fuse stage, eliminating residual drifts in the LLRS [28].

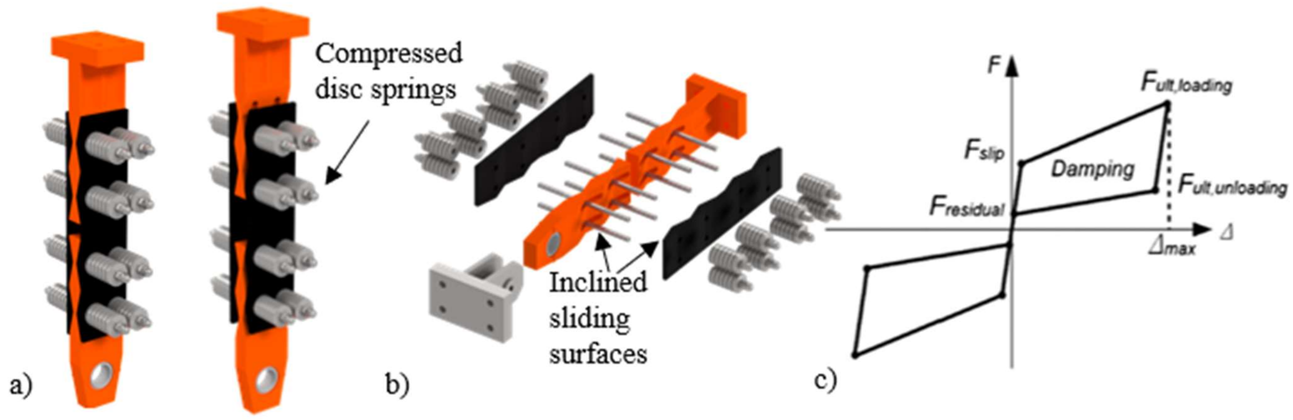

Figure 2 a)RSFJ Hold-down at rest and Extension b) Component Details c) Hold-down hysteresis

A coupled CLT wall LLRS incorporating energy dissipaters as couplers and RSFJ hold-downs offers a promising solution for taller mass timber structures (refer to Figure 1), with improved ductility, high damping, and no pinching. Residual drifts are also eliminated. However, the system is relatively new, and uncertainties related to test data, design methods, numerical modelling, and record-to-record variation must be carefully examined. This study reviews the various uncertainties associated with the LLRS and evaluates the system's robustness in terms of collapse prevention under a conservative range of uncertainties, based on available test data, literature, and FEMA-695 procedures. This paper forms a part of undergoing investigation, including full-scale tests and comprehensive analysis to determine seismic performance factors of the LLRS.

METHODOLOGY

Seismic behaviour factors for standardised LLRS include structural ductility factor μ, inelastic spectrum scaling factor kμ, and structural performance factor S_p in New Zealand standards [28]; force reduction factor R_d and overstrength factor R_o in Canadian standards [29]; and response modification coefficient R, overstrength factor Ω, and deflection amplification factor Cd in American standards [30]. These factors inherently account for uncertainties in the design process, test data, modeling, and earthquake records.

Seismic behavior factors for LLRS are derived either through extensive analysis and tests, or through experience with real-life earthquakes [31]. FEMA P695 provides a robust procedure for deriving these factors. In the absence of a methodology for evaluating uncertainties in newer systems in other standards, the FEMA P695 procedure is used to assess the system's robustness [31].

The flowchart in Figure 3 briefs the steps involved in the study, which are further described with the analysis on the selected archetype buildings.

System information

A part of gathering system information requires evaluating the uncertainties associated with the design process of the components, sub-assemblies, and complete assembly of the LLRS, as well as assessing the uncertainties in the test data for both comprehensiveness and variations.

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.

International standards and literature [24] [32] [33] [34] [35] have extensively evaluated and tested design methods for CLT walls and their connections at the component level, covering related uncertainties. Additionally, the performance of LLRS with CLT walls is established to be governed by the hold-downs, with the wall itself behaving rigidly [5] [3] [4] [24] [25] [26].Further, capacity design is specified for the design of CLT and its connections to limit the non-linearity to the hold-downs and couplers. The hold-downs and energy dissipating couplers are designed to published standards and have undergone 100% testing as part of the manufacturer's process, providing high confidence and comprehensiveness in the LLRS design process.

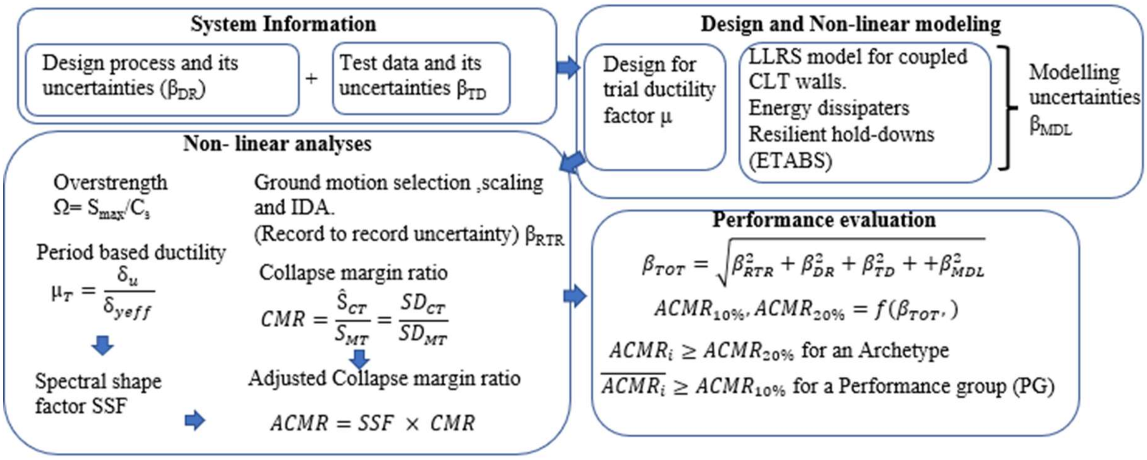

Figure 3 Uncertainties in collapse as per FEMA P695

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 have been added to the design process [4] [7], making it more comprehensive. The displacement capacity of the LLRS is specified to be minimum 1.3 times the MCE drift calculated using Displacement based design-Acceleration displacement response spectra (DBD-ADRS).

The design process has been established to provide reasonable safeguards against unanticipated failure modes. However, due to the limitations of the linear and non-linear static design methods, the process may be rated as medium to high in completeness and robustness. Therefore, 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, to ensure a conservative evaluation of the LLRS, a medium to fair design process uncertainty is chosen with a β_{DR} value of 0.35 (refer to Table 3-1 of FEMA-P695 [31]) for this study.

Test data and uncertainties

Component testing of CLT walls as an LLRS component has been established [1] [4] [36] [37] [38]. Further, CLT is being manufactured internationally adhering to published standards, minimising uncertainties.

RSFJ has been tested to provide symmetrical response in both directions with stable hysteresis at different slip loads [34] [39] [40] [41] [42]. The experiments and analytical predictions were within 5% and quasi-static tests with RSFJ hold-downs demonstrated repeatable hysteresis without degradation. RSFJ was tested on scaled and full-scaled timber braces following AISC 341 protocol and the experimental results and predictions showed excellent correlation with low variability [34]. High-frequency dynamic tests on RSFJ component also indicated stable hysteresis and self-centering with 5% variation at ultimate loads. Full-scale, bi-directional testing of a rocking CLT wall with RSFJ hold-downs verified the robustness of devices after out of plane actions were induced and the system depicted stable hysteresis with self-centering. Therefore, RSFJ consistently showed superior and reliable seismic performance.

Energy dissipaters chosen as couplers are friction dampers confirming ASCE-07 and ASCE-41 [30],[36], limiting variation to 15%, accounting for ageing, prototype testing, and manufacturing. However, proprietary friction dampers can have variations limited to 5% of the designed displacements and forces.

Full-scale shake table tests by Blomgren et al. [13] and Pie et al. [43] establish the diaphragm connection with rocking walls and performance of the mass timber gravity frame with or without PT cables. These tests also confirm the compatibility of the CLT wall with the gravity frame, reducing uncertainties related to test data. The performance of coupled mass timber walls, including CLT walls, has been tested and established by various researchers [13] [22] [19] [23] [43] - [44], however these walls were either PT walls or non-self-centering conventional walls. 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 studies discussed have provided high-quality test data, instilling confidence in the system's parameters. However, full-scale tests of coupled CLT walls with self-centering hold-downs for taller structures are still needed, resulting in a medium to high completeness rating. Thus, the test data uncertainty (β_{TD}) value of 0.1 to 0.2 should be considered for the LLRS. Nevertheless, a high β_{TD} value of 0.35 is selected to conservatively verify the robustness of the system.

Design and non-linear modelling

The study analysed a mixed-use building plan comprising 6, 8, and 10 storeys, with dimensions of 14.7 x 22.8 meters. The plan consists of 5 LLRS grids, each with two coupled walls. The designed wall length varies depending on the number of storeys of the archetype, with lengths of 3.5, 4.5, and 5.5 meters for 6, 8, and 10 storeys, respectively. However, to avoid impractical CLT spans, the 10 storey building archetype was also designed with a single wall (5.75m length) coupled with gravity boundary columns. The wall thickness is kept same at 315 mm (7 layers). The height of the first storey is 4.5 meters, while the subsequent storeys have a height of 3.2 meters. The modal time-periods for 6, 8, and 10 storeys are 0.64 to 0.86 seconds, while for the single wall coupled with gravity column is 1 second. The seismic weight of the first, second, and third to sixth/eighth/tenth storeys are 1306, 1137, and 879 kN, respectively. The diaphragm is assumed to be cross-laminated timber (CLT), and the gravity loads category is low.

The study involved the design and evaluation of four different coupled walls. The first three wall-wall coupled walls were designed and evaluated for a New Zealand scenario with a hazard factor Z of 0.3 and soil type D. On the other hand, the coupled wall-column LLRS was designed and evaluated for a Canadian scenario. The hazard for Victoria site class E with Sa values ranging from 0.08 to 1.36 for Sa(10) and Sa(0.2), respectively was chosen. The equivalent static method was used to size the walls and dampers for both the NZ and Canadian scenarios. A ductility (μ) of 3 was assumed for the NZ scenario, while an R_d value of 5 was considered for the Canadian scenario. The R_o value of 1.3 was considered pertaining to reliable secondary fuse mechanism of the RSFJs [42].

The force capacity of the damper was calculated based on static equilibrium, considering the coupled walls were horizontally constrained by diaphragm but free to rock. The energy dissipator (coupler) force capacity was set to prevent lifting of the coupled wall from the base, with a coupler slip force set at half the ultimate force of the hold-downs. This results in Moment sharing ratio of 0.25 between coupler and hold-downs for wall–wall coupling, while 0.33 between coupler and hold-downs for wall-column coupling. This is below the coupled wall criteria of 0.66 as per NBCC 2015 [29], but was adopted to ensure self-centering.

The Hold-downs (RSFJs) were designed with a displacement capacity of 1.3 times the MCE demands from the ADRS curve, and the coupler displacement accommodated the secondary fuse displacement capacity (1.8 to 2.2 times additional capacity) of the RSFJs. Other design considerations were kept following Agarwal et al. 2021 [7].

The CLT walls and its connections were capacity designed to suppress any un-simulated failure modes. The CLT walls were simulated using effective Young's modulus and shear modulus based on the CLT handbook [33]. The numerical model was validated by comparing the static load displacement with analytical equations. This CLT modelling technique used has already been established by researchers [36], [45]. 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 [4], [13], [24], [5], [3], [25], [26]. Non-simulated failure modes were assessed by checking demands from the non-linear analysis on elastic members.

To model the LLRS, commercially available software ETABS [46] was used, and the RSFJs were modelled using a damper friction link element. The RSFJs can be modelled in any software that can simulate a flag-shaped hysteresis with single or combination of linear and non-linear spring elements. The accuracy of the model was verified by researchers [45] [47] [48], who found that it was accurate to the analytical model and test results within 5%. Moreover, the flag-shaped hysteresis behavior of the RSFJ is repeatable without degradation, making it more accurately modelled than other types of connections that degrade and pinch.

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 [49], [50], [51]. The modelling process and hysteresis are well-established and available in most research and commercial software.

In summary, the numerical model that can accurately capture the full range of structural behavior of the LLRS until collapse. Following Table 5-3 of FEMA P695 a low modeling uncertainty (β_{MDL}) of 0.1 is suggested for the LLRS; however, to retain conservatism in the study, a value of modeling uncertainty of 0.2 (β_{MDL}=0.2) is chosen.

Non-Linear analysis

The FEMA P695 methodology for the evaluation of uncertainties requires partial 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. The period-based ductility (μ_T) is the ratio of yield displacement to the ultimate displacement causing collapse. This should not be confused with the design ductility μ. It is important to note that the period base ductility of the structure evaluated in this study exceeds 3. To ensure a conservative approach, a value of 1.22 [30] is chosen for the wall-wall coupling archetypes, while a value of 1.25 is selected for the wall-column coupling archetypes.

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.

A set of 44 high-magnitude, far-field ground motions specified by FEMA-P695 was selected for this study. The records were normalised by their peak ground velocity to remove biases due to magnitude, source distance, source type, and site condition, while retaining frequency content and record-specific collapse mechanisms. Initially, the records were scaled to match the MCE spectra by setting the median spectral acceleration at the fundamental period of the archetype. Further scaling was performed to conduct IDA until the CMR value corresponding to the considered uncertainties was achieved. The FEMA P695 pegs the uncertainty for the ground motion records (β_{RTR}) as 0.4 for the US seismic scenario. However, for the NZ and Canadian seismic scenarios, a record set with uncertainties may need to be developed.

Performance evaluation

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

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

β_{TOT} defines the lognormal standard deviation λ_{TOT}, which is the product of four variables: variables λ_{RTR}, λ_{DR}, λ_{TD}, λ_{MDL}. These variables are defined as independent lognormally distributed variables with median values of unity and lognormal standard deviation parameters corresponding to the evaluated uncertainties (β_{RTR}, β_{DR}, β_{TD}, β_{MDL}).

The LLRS considered in this study is a state-of-the-art low damage system, providing high confidence in its performance. The range uncertainties related to design, test data, and modeling varies from 0.1 (superior) to 0.5 (poor), as per FEMA P695 (2009).

The study incorporates a conservative approach, with a calculated total system uncertainty of 0.667. However, it is important to note that the actual system uncertainty could range between 0.435 to 0.529. In order to achieve greater confidence in the implementation of the discussed LLRS with reduced uncertainties, a full assembly test to confirm the performance of coupled shear walls with self-centering hold-downs would be necessary and is planned by the authors.

The lognormal collapse probability is typically determined by $\hat{S}_{ct}$ and β_{RTR}, where $\hat{S}_{ct}$ represents the median collapse intensity and β_{RTR} reflects the dispersion in results due to record to record uncertainty. For this study, β_{RTR} is fixed at 0.4 based on various studies and its low impact on CMR values when combined with other uncertainties, in accordance with FEMA P-695.

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

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

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

The mentioned total system uncertainty value of 0.667, resulted in ACMR values of 2.3 for the studied archetypes. As a result, maximum IDA scaling factors of 3.1, 3.5, and 3.56 were applied to the 6, 8, and 10 storey coupled wall-wall archetypes, respectively (refer to Figure 4 and Figure 5). For the coupled wall-column archetype building, a maximum IDA scaling factor of 4.67 was used (refer to Figure 5).

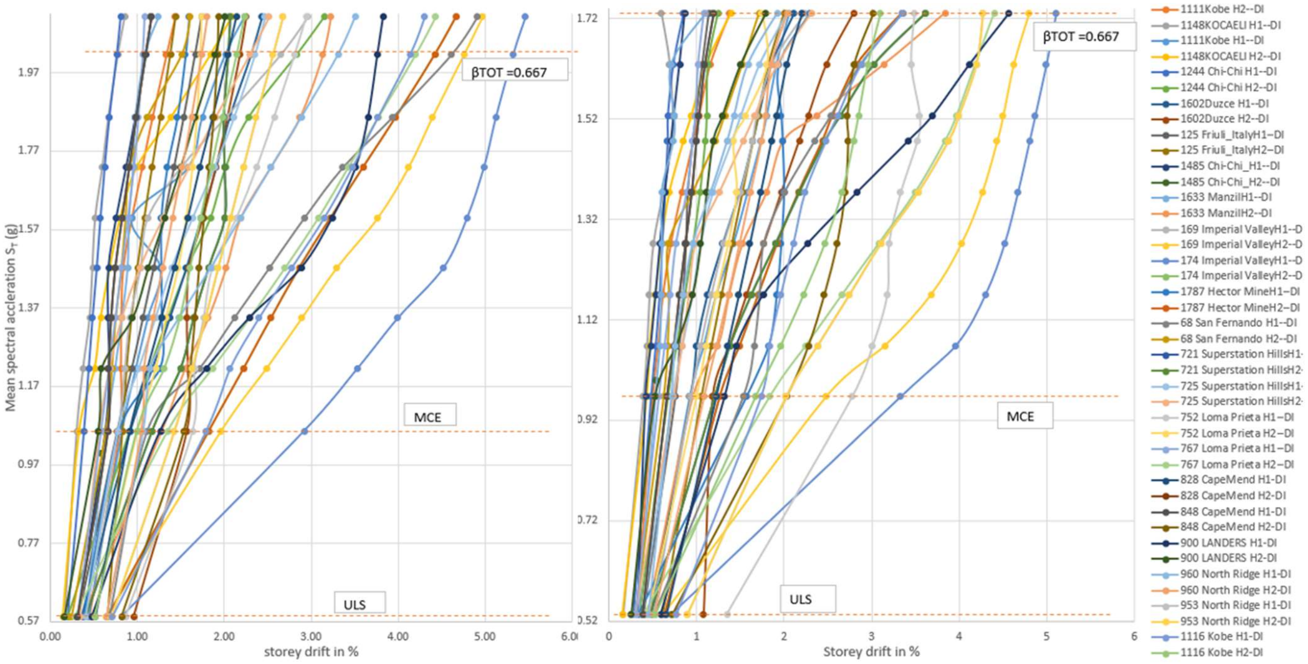

Figure 4 Collapse evaluation – IDA for 6-storey (left) and 8 storey (right) coupled wall-wall LLRS.

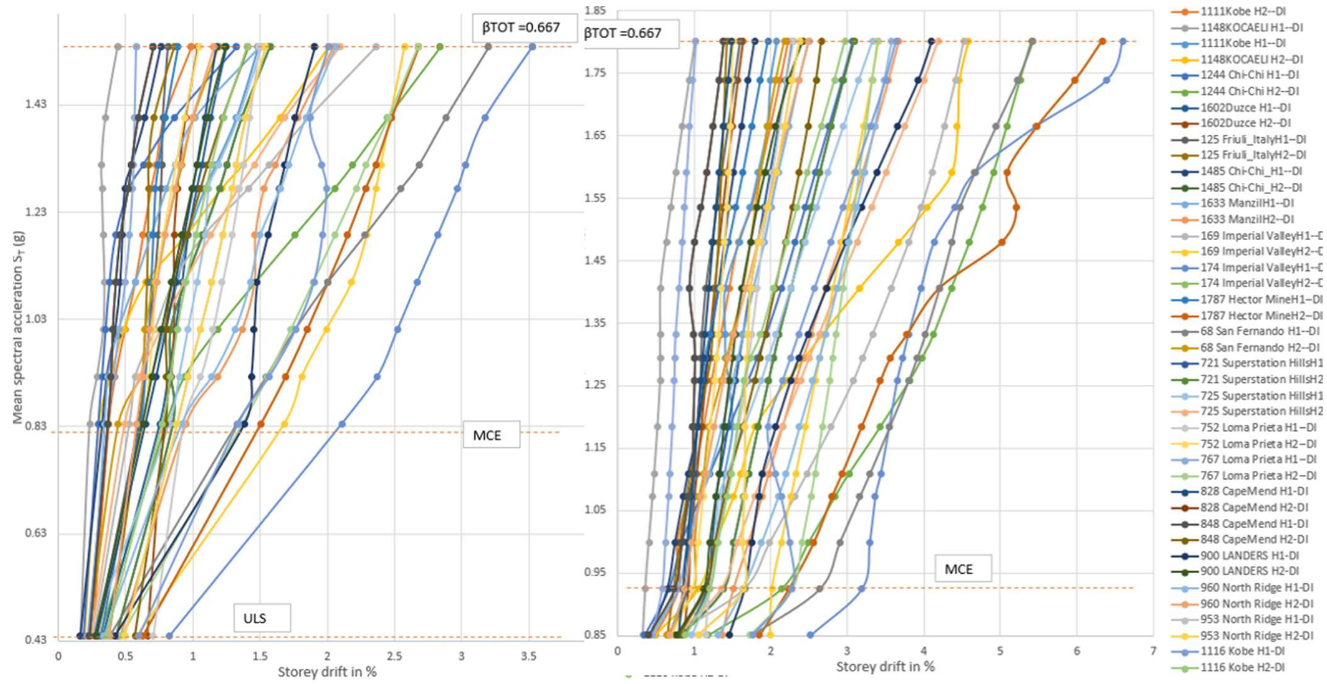

Figure 5 Collapse evaluation–IDA for 10-storey coupled wall-wall LLRS (left) and 10-storey coupled wall-column LLRS (right).

RESULT AND DISCUSSION

The IDA analysis was performed to meet the back-calculated ACMR values based on the uncertainties. The criteria of 5% drift for gravitational system compatibility [40] was included, and the results were limited to the range from spectral acceleration corresponding to MCE to the accelerations corresponding to required ACMR values. However, the analysis did not include the complete IDA analysis from low spectral acceleration to complete collapse in each ground motion. The linear portion of the analysis and higher acceleration values were not included in the graphs (refer to Figure 4 and Figure 5). Additionally, the LLRS considered in the study had a positive secondary stiffness, even in the secondary fuse stage of the hold-downs, which prevented a sudden slope change in the IDA curves after yielding (refer to Figure 4 and Figure 5). The secondary stiffness remained relatively constant from the first slip to the final collapse of the structure. Due to this unique secondary stiffness and the truncation of the IDA analysis, the shape of the IDA curves was considerably different from conventional IDA curves (refer to Figure 4 and Figure 5).

The analysis indicates that the structure exhibits high robustness at the acceptable ACMR values, which were determined using conservative values of uncertainties. This implies that the structure was capable of resisting most of the ground motions considered in the analysis, even at the conservative ACMR values. Moreover, it is important to note that the collapse probability was still lower than the 10% limit. This suggests that there may be potential to use higher force reduction factors for the LLRS, and further analysis may be required to verify these factors.

The analysis results revealed that for the coupled wall-wall archetypes, the mean moment of the records at the scaled MCE intensity was lower than the base moment capacity of the designed LLRS, indicating negligible moment amplification above the base due to higher mode effects. However, for the wall-column archetype, there was a mean moment amplification at the maximum scaled IDA value, which was offset by the moment amplification factor set in the design requirements, restricting the probability of collapse less than 10%.. For the wall-column archetype (Canadian scenario), the reduction factor was nearly twice that of the coupled wall-wall archetypes (NZ scenario), resulting in a scaling of the spectral accelerations to nearly twice the values for the NZ scenario. This finding is consistent with previous research showing that higher mode effects are amplified with increased spectral intensities..

The amplification of higher modes in the shear force was counterbalanced by the larger inherent shear capacity of the CLT walls restricting the probability of collapse less than 10%. However, the numerical models used in the study only considered the CLT wall's shear stiffness, while the shear keys were modelled as rigid toes. Further investigation is necessary to accurately model the shear key stiffness and the shear springs along the height of the wall.

The design drifts at ULS and MCE levels using the DBD-ADRS were found to be conservative compared to those obtained from the analysis. With all the uncertainties considered, the probability of exceeding a 2.5% drift was found to be less than 10%, which is in compliance with the drift requirements of NBCC 2015. This confirms the robustness of the design procedure using DBD-ADRS. Furthermore, no correction was required for the Equivalent Hysteresis Damping (EVD) for the selected LLRS.

CONCLUSION

The study used FEMA P695 methodology to evaluate four archetype buildings, including three with coupled walls of 6, 8, and 10 stories and one with wall column coupling, all with a high aspect ratio in the mid to high-rise range. The analysis considered uncertainty related to test data, design process, modelling, and earthquake record. The LLRS comprised of coupled CLT walls with self-centering hold-downs and an energy dissipater as a coupler. A conservative value of total system uncertainty of 0.667 was used to assess the LLRS, which showed robustness against collapse with less than 10% probability of collapse at the MCE level of earthquakes. The probability of exceeding 2.5% drift was also less than 10%.

The LLRS demonstrates the potential to design for higher ductility, even with conservative values of uncertainties. However, more realistic values of uncertainties, in line with available data, could yield more favourable results. The authors are currently performing a comprehensive analysis to derive seismic behaviour factors for the LLRS.

The DBD-ADRS design method accurately predicted the design drift without the need for correction in the equivalent hysteretic damping. However, the base shear amplification due to higher mode effects still requires further evaluation. This amplification was counterbalanced by the larger inherent shear capacity of the CLT walls.

Overall, this study highlights the potential of using coupled CLT walls as an effective seismic resistant system, and suggests the need for a more comprehensive analysis to derive seismic behaviour factors for this type of system.

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