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Seismic Assessment of a Hybrid Light Wood Frame Structure Connected to a Balloon-Type CLT Core

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ABSTRACT

The National Building Code of Canada requires the lowest seismic force modification factors (R_dR_o) of the subsystems be used for the design of a hybrid building if the subsystems are rigidly connected, which may be conservative if a ductile connection is used. This paper presents a numerical study on the effect of connection ductility on the seismic performance of a hybrid building consisting of light wood frame shear walls and a balloon-type CLT core. The two subsystems were connected using self-tapping screws (STS) inserted at 45°, 90°, and mixed angles (45° + 90°). Pure light wood frame structures and pure CLT structures were also analyzed as reference cases. One-, four- and six-storey archetypes were designed with trial R_dR_o factors. The OpenSees software was used to develop a 2D numerical model for each archetype. The R_dR_o of the five analyzed cases was evaluated following the Canadian Construction Materials Centre guideline. Incremental dynamic analysis (IDA) was carried out using the 22 FEMA P695 far-field ground motions. The results show that Rd = 2 and Ro = 1.5 are acceptable for cases of pure CLT structures, and hybrid structures connected using STS inserted at 45° and 90°. The hybrid buildings connected using STS inserted at mixed angles (45° + 90°) can be assigned with Rd = 2.5 and Ro = 1.5. The archetypes designed with Rd = 3 and Ro = 1.7 are deemed satisfactory for pure light wood-frame structures.

Keywords: Seismic force modification factors, Light wood frame shear wall, CLT core, Connection ductility, Hybrid building

INTRODUCTION

Most low-rise (up to 4 storeys) residential buildings in North America are constructed with light wood frame (LWF) systems [1]. LWF structures are characterized by their flexibility and energy absorption capabilities through numerous nailing connections, enabling them to withstand earthquakes well in the past [2]. While the National Building Code of Canada [3] allows the construction of up to 6-storey buildings with LWF walls as the lateral load resisting system (LLRS), the use of LWF construction in mid-rise wood building sector remains challenging due to restrictions on architectural design and lack of stiffness in wind design [4]. To address these challenges, a program called TF2000 [5] had been initiated in the UK. In that project, dynamic characteristics of a full-scale six-storey LWF structure were tested under forced and ambient vibrations. The results showed that addition of a staircase to bare wood frame prototype significantly increased the translational stiffness. There have been many other efforts to combine LWF construction with stiffer structural systems to compensate for the low lateral rigidity [6-7].

One possible solution to the lack of lateral rigidity of multi-storey LWF system is to combine the CLT core with LWF construction. Hybridizing the two subsystems could potentially result in improved stiffness, strength, ductility, thus achieving the desired design targets. CLT walls behave relatively rigid thus the connections are the only source of ductility [8]. When combining two structural systems, the links between the two subsystems could also contribute to the energy dissipation of the whole system. In current practice, the core (stairway and elevator shaft) is usually built structurally separate from the LWF system. NBCC [3] recommends using the lowest value of the R_dR_o of individual systems when designing a hybrid structure. This means that the higher energy dissipation capacity of the more ductile system would be ignored in the design, which may be a conservative approach. This study investigates the effect of different connection ductility on the seismic response of a hybrid building system. Structural models are designed with self-tapping screw (STS) connections inserted at 45°, 90°, and mixed angles (45° + 90°) and Incremental Dynamic Analysis (IDA) [9] was run to assess if the proposed R_dR_o provides sufficient safety margin.

Only a handful of studies attempted to evaluate the response modification factors of hybrid timber structures. Zhou et al. [10] studied multi-story hybrid light wood frame buildings connected to a masonry core assuming different resisting ratios of the two sub-systems and the connections between them. The seismic force modification factors and the fundamental periods were investigated. The authors found that the relative stiffness of the wood, masonry and connection systems and the ultimate deformation of the sub-systems influenced the failure mode. They proposed that a larger R_dR_o factor than the lowest values of the two systems could be used to design the hybrid structure. Follesa and Fragiacomo [11] studied a LWF/CLT building with CLT floor diaphragms with LWF walls and CLT walls as LLRS. The ductile connectors in this research were CLT-to-CLT angle brackets and hold-downs. The anchor bolts and screwed LWF wall-to-foundation and LWF wall-to-floor connections were modeled with linear springs. The contribution of CLT wall and LWF wall to lateral resistance varied among archetypes. They proposed and verified the analytical formulation for determining the ductility-related force modification factor (R_d) of hybrid systems. Chen et al.[12] developed empirical equations of ductility ratio, μ, and R_d based on the strength ratio of the individual LLRSs for systems with different ductility levels. Tesfamariam et al. [13] evaluated a hybrid system of CLT walls and reinforced concrete beams. They performed FEMA P695 [14] procedure using Canadian seismicity and design factors to quantify the ductility-related force modification and found that the $R_dR_o = 2 \times 1.5$ were acceptable for this hybrid system.

A thorough performance assessment is needed to get a better understanding of the influence of connection ductility on the performance of hybrid systems. Herein, the experimental results of a prior project on self-tapping screw (STS) connections between LWF construction and CLT wall panels were employed to compare the seismic response of hybrid LLRS buildings. The Canadian Construction Material Center (CCMC) [15] procedure is used to quantify the force modification factors for the considered archetypes.

EVALUATION OF SEISMIC PERFORMANCE

The FEMA P695 [14] is a procedural methodology which establishes the seismic response parameters for a new lateral load resisting system. It requires that archetypes cover all expected range of structural and geometrical parameters which is usually time-consuming. Herein, the newly developed CCMC [15] guideline based on FEMA P695 which suits NBCC [3] was followed. According to this simplified procedure, the evaluation of seismic performance was carried out with the following steps:

- There were 15 archetype structures (five configurations designed with three building heights) designed using Equivalent Static Force Procedure (ESFP) with initial estimates of R_dR_o.
- Nonlinear springs were used to simulate energy dissipative elements that include STS connections between the two subsystems, LWF shear walls, CLT-to-foundation hold-downs. The deformation capacity for yielding elements was specified as the displacement when the load drops to 80% of maximum resistance [16]. The non-dissipative elements were designed using capacity design rules (low-ductility STS connections, shear connectors and CLT panels).
- The nonlinear dynamic analysis was performed using 22 far-field earthquake records recommended by FEMA P695 and scaled to the uniform hazard spectrum (UHS), a level of ground motion with 2% change of being exceeded in 50 years according to NBCC [3] The maximum inter-storey drift ratio limit of 2.5% specified in the NBCC [3] was adhered to for all responses to motions that were scaled to 100% of UHS.
- If the design satisfied the requirements under the 100% UHS intensity level ground motion, a second series of nonlinear dynamic analysis was conducted using the ground motions scaled to 200% of the UHS. In other words, the acceptance criteria of CCMC are consistent with an adjusted collapse margin ratio of 2 in the FEMA P695 procedure.
- Failure of the hybrid structures was defined as either the dynamic instability or deformation capacity (80% drop down displacement) exceedance happened. The system failure is also defined as when more than 50% of ground motions results in an unacceptable response.
- If all performance criteria were met at 100% and 200% of the UHS, then R_dR_o was accepted for the single archetype. Otherwise, the system was reanalyzed with a lower R_dR_o and the procedure was repeated.

IDENTIFICATION OF CASE STUDY CONFIGURATION

The first step is to establish archetypes which are representative of the possible design configurations. The properties of the connection between the LWF construction and CLT core influence how the two subsystems interact under the seismic load. Therefore, three types of connections were considered for the hybrid archetypes: STS connections inserted at 45°, 90°, and mixed angles (45° + 90°). As reported in the experimental study [17], 45˚ STS, 90˚ STS and mixed-angle STS connections have a ductility ratio (μ) of 3.3, 3.7 and 37.7, respectively. Since the 45˚ STSs have a small ultimate deformation = 4.8 mm, they were characterized as non-dissipative elements. 90˚ STSs have large ultimate deformation = 30.3 mm, so they were regarded as energy-dissipative connections. The mixed-angle STSs have both high ductility ratio and large ultimate displacement = 41.7 mm, so they are identified as high-ductility connections. There were three main configurations of LLRSs considered in this project: (1) pure LWF wall construction (Case A); (2) hybrid LWF-CLT building with different types of connections (case B: 45° STS connections, case C: 90° STS connections and case D: mixed-angle STS connections); (3) pure balloon-type CLT core wall building (Case E) which are presented in Figure 1. Each configuration was designed in 1, 4 and 6-storey archetypes to cover the range of low-rise to mid-rise LWF construction. The standard LWF shear wall tested by Ni et al. [18] was used as LWF walls in this study. Hold-downs with *Φ*20 mm dowels tested by Brown and Li [19] were used for CLT core wall connections to the foundation. The CLT core wall was detailed with capacity protected shear connectors to control the sliding of the wall. Conventional angle brackets test data [20] was used to represent shear connectors. The behavior of connections were derived from reversed-cyclic tests on 10 mm STS lumber-to-CLT connections loaded along the minor axis of 5-layer CLT panels [17].

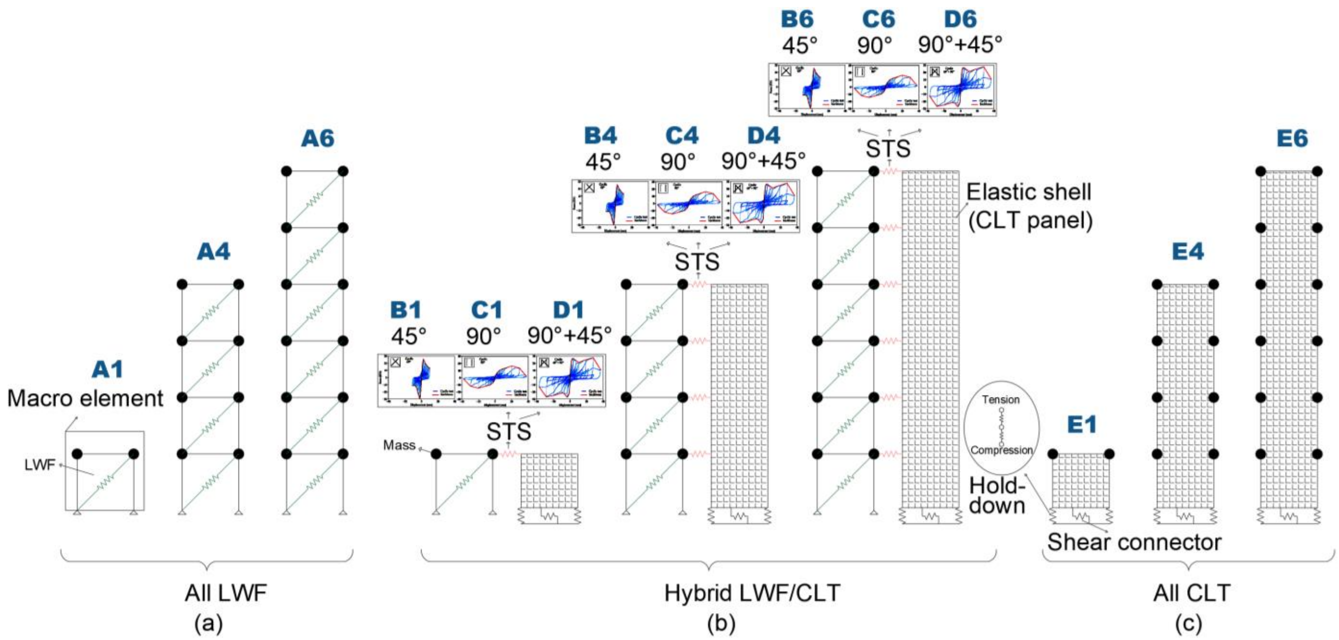

Figure 1. Archetype cases: (a) Pure LWF construction; (b) Hybrid LWF/CLT cases; and (C) Pure CLT core wall as LLRS.

ARCHETYPE DESIGN

Five different building configurations were considered in this study. All these models were residential buildings that had the same plan layout of 33 × 26 m (Figure 2) and floor height of 2.8 m. The direction of earthquake load under consideration was along the East-West direction. The CLT core walls in case A were assumed to be structurally separated from the LWF system, as a result they were not included in the finite-element model. But in hybrid configurations, CLT walls are linked to the light wood frame structures. In case C, the CLT cores are the only system resisting the lateral loads. Since the floor plan was symmetrical, only a quarter of layout was used in finite element modeling and mass calculation (containing one of the CLT core walls and surrounding LWF walls). The building was designed for a lumped dead load of 1.8 kPa and 0.95 kPa for floors and roof, respectively. A snow load of 1.08 kPa was also considered in the calculation of the roof seismic weight. The buildings were assumed to be in Vancouver on site soil class C. According to NBCC [3] the design spectral acceleration for periods of 0.2 s, 0.5 s, 1 s, 2 s and 5 s were 1.09 g, 0.876 g, 0.508 g, 0.309 g, and 0.087 g, respectively. A primary ESFP was carried out using fundamental periods (T_a) calculated based on NBCC [3] ($T_a = 0.05h^{3/4}$, where h is the building height in meter). Using the stiffness derived based on the Equivalent Energy Elastic-Plastic (EEEP) method [21], the analytical period was obtained. All analytical periods were more than twice the code's empirical value ($2T_a$) which are presented in Table 1. Since $2T_a$ is the cut off point for strength design required by NBCC [3], $2T_a$ was adopted as the final design period with no further iteration. The corresponding design spectral acceleration of the final design for 1, 4 and 6-storey buildings were 1.08 g, 0.79 g and 0.64 g, respectively.

Table 1. Fundamental periods of archetypes.

Period (s)	**1-storey**	**4-storey**	**6-storey**
Ta (NBCC)	0.11	0.31	0.41
2 × *Ta*	0.22	0.62	0.82

Notes: [1] Period calculated based on mechanical properties and EPP curve.

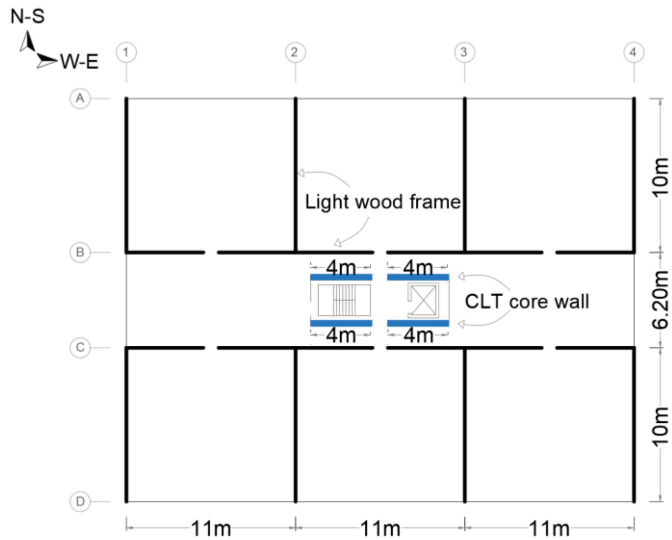

Figure 2. Floor plan.

The ESFP incorporates the inelastic behavior (ductility-related factor) and the dependable portion of reserved strength (overstrength-related factor) by reducing elastic base shear by R_dR_o. The NBCC [3] adopted the ductility factor (R_d) for the design of platform construction using CLT shear walls and LWF construction as 2 and 3, respectively. However, no seismic force modification factor is specified for designing balloon-type CLT walls in the NBCC [3]. The final values in the iteration process of quantifying R_d factor were presented in this paper. Case A was designed with $R_d = 3$. Case B, C and E were designed with $R_d = 2$, case D was designed with $R_d = 2.5$.

The overstrength factor takes into account the actual strength of materials, i.e., the effect of discrete member sizes, the increased strength due to nonstructural elements, strain hardening, difference between nominal and factored resistances and redistribution of internal forces in the inelastic range [22]. R_o was set to be 1.7 for the configurations with only LWF construction (case A). As suggested by CCMC [15] for balloon-type CLT and other hybrid configurations (case B, C, D and E) $R_o = 1.5$ was adopted. The final accepted seismic modification factors based on CCMC guideline are listed in Table 2.

Table 2. Seismic force modification factors of archetype cases.

Configuration	**Description**	R_o	R_d
Case A	LWF wall construction	1.7	3
Case B	Hybrid LWF/CLT with 45° STS connections	1.5	2
Case C	Hybrid LWF/CLT with 90° STS connections	1.5	2
Case D	Hybrid LWF/CLT with mixed-angle STS	1.5	2.5
Case E	CLT core wall system	1.5	2

The LWF system on each storey was assumed to be a macro-element that represents all the LWF walls in that storey. According to wood design manual [23], the factored shear strength of LWF shear wall is equivalent to half the ultimate lateral load determined from testing. The reversed cyclic test result of wall 3r' (2.44 m × 2.44 m) in Ni et al. [18] was used to calibrate the LWF wall macro-element. This wall has an ultimate deformation of 84 mm (3.5% drift ratio). Based on the recorded maximum resistance of 59.8 kN (24.5 kN/m), the design shear force of the 2.44 m long LWF wall was determined to be 29.9 kN, which corresponds to half of the maximum resistance [10].

The design strength of hold-downs (HD), shear connectors (SC) and connection (STS) between the two subsystems were obtained based on statistics of experiments and literature-based recommendations for capacity design principles. The capacity-based design rules have been integrated into seismic design guidelines to ensure the expected energy dissipation occurs in ductile elements and brittle failure modes are avoided. This is accomplished through multiplying the design resistance of non-dissipative connections with a capacity adjustment factor (γ) derived based on dissipative connections (the ratio of 95th percentile of dissipative connection to its design resistance). However, this factor is commonly known as overstrength factor [24], but due to presence of R_d (overstrength-related force modification factor) in this paper and to avoid confusion, we will use the term "capacity adjustment factor" to refer to γ throughout the remainder of the paper. As per CSA O86-19 [25], non-dissipative elements must remain elastic when the dissipative elements reach their 95th percentile ($R_{95^{th}}$) of ultimate strength. The $R_{95^{th}}$ of elements were found through the mean of test's maximum resistance ($R_{max,mean}$) and coefficient of variation (CoV), assuming a normal distribution in test results. Subsequently, the design value of connections (R_{design}) was determined as the ratio of $R_{95^{th}}$ to the corresponding γ factor. Table 3 presents the design strengths of connections' tests before scaling and capacity protection rules were applied.

Table 3. Design strengths of connections and CLT core elements.

Element	**Ductility ratio (μ)**	**Ultimate deformation (mm)**	**$R_{max,mean}$ (kN)**	**CoV (%)**	**$R_{95^{th}}$ (kN)^1**	γ	**R_{design}^{2} (kN)**
45° STS (Case B) [17]	3.3	4.8	35.5	18.0	46.0	1.6 [24]	28.8
90° STS (Case C) [17]	3.7	30.3	22.4	5.0	24.2	1.6 [24]	15.1
45° + 90° STS (Case D) [17]	37.7	41.7	39.0	11.0	46.0	1.6 [24]	28.8
Hold-down (HD) [19]	23.2	42.5	268.5	1.2	273.8	1.5 [19]	182.5
Shear connector (SC) [20]	7.3	41.3	13.4	5.0	14.5	1.3 [24]	11.2

Notes: ^{1} $R_{95^{th}} = R_{max,mean}\,(1 + 1.645 \times CoV)$; ^{2} $R_{design} = R_{95^{th}} / \gamma$

The subsystems were designed to equally share the design base shear. Therefore, specified lateral earthquake force of NBCC was split in half and applied to both the LWF system and the CLT core wall. This approach makes the design forces of connections to be the same as the NBCC load distribution pattern. Table 4 shows the design forces that were determined through Equivalent Static Force Procedure (ESFP). The design values embodied the capacity rules for non-dissipative elements. In order to ensure that the design capacities were appropriately accommodated, the elements in the numerical model were subjected to scaling to calibrate their design strengths (R_{design}) with the forces shown in Table 4.

The dissipative elements of case E (pure CLT) and case B (hybrid building with low-ductility connections) are the hold-downs. Therefore, to ensure the yielding of hold-downs are given priority and CLT remains elastic γ of hold-down was applied to the NBCC's seismic design forces of CLT panels. Because shear connectors are designed to have limited sliding behavior, γ of hold-down was also applied to the NBCC's base shear and used as the force to scale shear connectors. γ of hold-down was set 1.5 which were recommended based on characteristic experimental strengths for dowelled CLT hold-downs [19].

Another primary energy dissipative element in hybrid configurations with 45°, 90° STS connections (Case C and D) are the STS connections between the LWF and CLT core systems. As recommended by Gavric et al. [24], a capacity adjustment factor of 1.6 was chosen for hybrid cases, as the value considered for panel-to-panel screw connection test were reported 1.6. The 45° STS is categorized as an element with limited ductility and is capacity protected to behave rigidly. Therefore, a capacity adjustment factor of 1.3 was chosen to design the 45° STS between the subsystems.

The CLT panels were designed to meet CSA O86-19 [25] requirements with a width of 4 m. The end distance between the hold-downs and the corner of wall was assumed to be 40 cm. For case E, the 1, 4 and 6-storey archetypes were detailed with 3-ply (105 mm thick), 5-ply (175 mm) and 7-ply (245 mm) grade E1 CLT panels, respectively. The 1, 4 and 6-storey hybrid archetypes were detailed with 3-ply (105 mm thick), 3-ply (105 mm thick) and 5-ply (175 mm thick) grade E1 CLT panels, respectively. Since the panel width plays an important role in rocking behavior of the CLT core, wall width was kept consistent across all archetypes for the sake of comparison.

Table 4. Design forces (kN) for LWF walls, STS connections, hold-downs (HD) and shear connectors (SC) of each archetype.

	A1	**B1**				**C1**				**D1**				**E1**	
Storey	LWF	LWF	STS$^{*(1.3)}$	HD	SC$^{*(1.5)}$	LWF	STS	HD	SC$^{*(1.6)}$	LWF	STS	HD	SC$^{*(1.6)}$	HD	SC$^{*(1.5)}$
1	55.6	47.3	61.4	41.4	70.9	47.3	47.3	41.4	70.9	37.8	37.8	33.1	56.7	82.7	141.8
	A4	**B4**				**C4**				**D4**				**E4**	
Storey	LWF	LWF	STS$^{*(1.3)}$	HD	SC$^{*(1.5)}$	LWF	STS	HD	SC$^{*(1.6)}$	LWF	STS	HD	SC$^{*(1.6)}$	HD	SC$^{*(1.5)}$
4	68.8	58.5	76.0	-	-	58.5	58.5	-	-	46.8	46.8	-	-	-	-
3	144.9	123.2	84.1	-	-	123.2	64.7	-	-	98.5	51.8	-	-	-	-
2	195.6	166.3	56.1	-	-	166.3	43.1	-	-	133.0	34.5	-	-	-	-
1	221.0	187.9	28.0	468.8	282	187.9	21.6	468.8	281.8	150.3	17.3	375.0	225.4	937.6	563.5
	A6	**B6**				**C6**				**D6**				**E6**	
Storey	LWF	LWF	STS$^{*(1.3)}$	HD	SC$^{*(1.5)}$	LWF	STS	HD	SC$^{*(1.6)}$	LWF	STS	HD	SC$^{*(1.6)}$	HD	SC$^{*(1.5)}$
6	59.2	50.3	65.4	-	-	50.3	50.3	-	-	40.2	40.2	-	-	-	-
5	131.9	112.1	80.4	-	-	112.1	61.8	-	-	89.7	49.5	-	-	-	-
4	190.1	161.6	64.3	-	-	161.6	49.5	-	-	129.3	39.6	-	-	-	-
3	233.8	198.7	48.2	-	-	198.7	37.1	-	-	159.0	29.7	-	-	-	-
2	262.9	223.5	32.2	-	-	223.5	24.7	-	-	178.7	19.8	-	-	-	-
1	277.4	235.8	16.1	859.2	354.0	235.8	12.4	859.2	353.7	188.6	9.9	687.4	283.0	1718	707.4

Notes: $^{*(\gamma)}$ These elements are capacity protected and the applied capacity adjustment factors (γ) are stated in the parenthesis.

INCREMENTAL DYNAMIC ANALYSIS

Finite element modeling

Simplified numerical models have been proven to reproduce responses that matched the experimental results [26]. As a part of the CUREE-Caltech wood frame research project [27], a program called Seismic Analysis of Wood frame Structures (SAWS) [28] was developed which has the ability to model the building as a two-dimensional system. A 2D numerical model was developed in OpenSees [29]. Each storey of the LWF construction was simulated with three rigid truss elements and one diagonal nonlinear spring. The SAWS [30] model was adopted to simulate the hysteresis performance of this macro-element. The parameters of SAWS model are listed in Table 5.

Table 5. SAWS model parameters for LWF shear wall macro-elements.

Parameter	***S0***	***DU***	***α***	***ß***	***R1***	***R2***	***R3***	***R4***	***F0***	***FI***
Value	7.0	42	0.7	1.1	0.065	-0.09	1.22	0.03	68	8

As shown in Figure 1, elastic shell elements were used for CLT panels because nonlinearity is only expected to happen at the joints. Since hold-down only resists uplift, DowelType uniaxial properties [31] in OpenSees were assigned to a nonlinear spring (zero length element) in tension. The contact compression of the wall edge to the foundation was modelled with another spring using elastic-no-tension material with an elastic modulus of 1500 MPa as recommended by Sun et al. [32]. The shear connectors (CLT-to-foundation) and CLT-to-LWF connection were also modelled with the DowelType material and the parameters of the model are listed in Table 6. To better simulate the experimental hysteresis, only the mixed angle connection (45°+90° STS in case D) was modelled with piece-wise linear envelope curves, and all the other connections were simulated with Bezier curve as their backbone.

The verification of numerical results with reversed cyclic loading test on each element is shown in Figure 3. The total energy dissipated during the cyclic tests is consistent with the energy observed during the tests. The 45° STS, 90° STS, mixed angle connection, hold-down, shear connector and LWF wall failed at 4.8 mm, 30.3 mm, 41.7 mm, 42.5 mm, 41.3 mm and 84 mm, respectively, based on 80% drop down of the peak resistance.

Table 6. Model parameters of connection, hold-down and shear connector.

Parameters (Bezier curve)	**STS 90°**	**STS 45°**	**Hold-down**	**Shear connector**	**Parameters (Piece-wise linear)**	**STS 45°+90°**
F_{I0}	1.3	4	2.5	0.53	F_{I0}	1.1
K_{p0}	0.97	10.4	0.6	0.75	K_{p0}	2.8
R_{u0}	3.1	1.1	1.1	2.3	R_{u0}	0.9
C	0.8	1.1	0.6	0.8	C	0.7
β	1.1	1.04	1.05	1.12	β	1.08
γ	1	1	1	1	γ	1
η	0.1	0.14	0.07	0.14	η	0.08
D_y	3.4	1.15	0.1	2.1	D_y	0.85
α_p	1.1	1.29	0.2	0.9	α_p	1.15
α_u	0	0.1	0.06	0	α_u	0.03
α_r	0.85	0.7	0.68	0.31	α_r	0.7
D_{b1} (F_{b1})	2.1 (11)	0.5 (17)	0.3 (255)	3 (8.1)	D_1 (F_1)	1.7 (37.5)
D_{b2} (F_{b2})	7 (12.1)	1.1 (28)	5 (260)	5.9 (10)	D_2 (F_2)	10 (25)
D_c (F_c)	26 (23)	2.5 (35)	35 (269)	24.5 (14)	D_3 (F_3)	34.2 (39)
K_d (D_u)	1.5 (30.5)	2.5 (4.8)	5 (42.5)	0.1 (41.3)	D_4 (F_4)	43 (30.5)

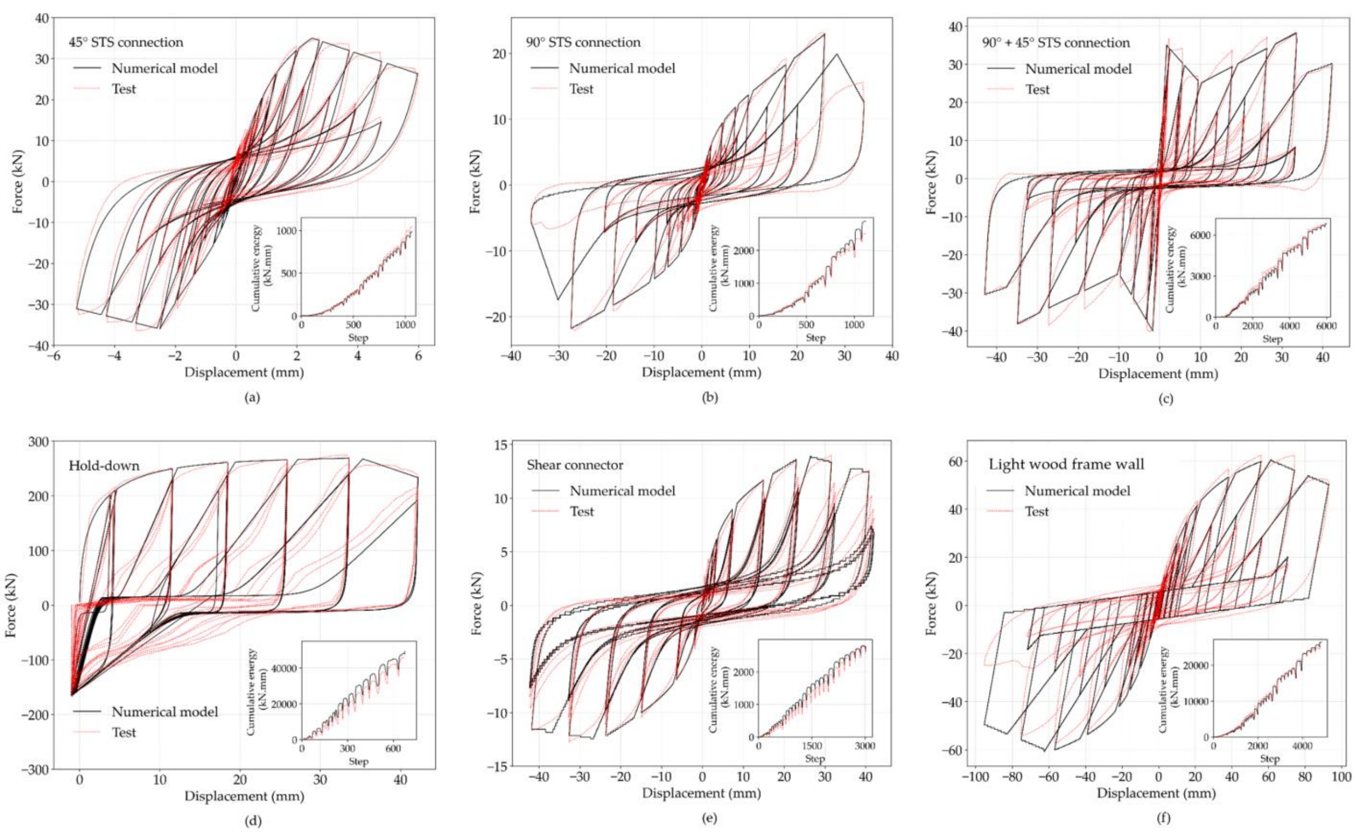

Figure 3. Comparison of hysteresis loops of numerical model and tests: (a) 45° STS connection; (b) 90° STS connection (c) 90° + 45° STS connection; (d) Hold-down; (e) Shear connector; and (f) LWF wall.

Nonlinear dynamic time history analysis

To perform non-linear time history analysis, each archetype was subjected to a single component of the 22 far-field earthquake records suggested by FEMA P695 [13] guidelines. First, the ground motions were normalized with respect to median peak ground velocity. Then, the records were collectively scaled to the NBCC 2020 [3] uniform hazard spectrum (UHS) of Vancouver. Figure 4 compares the median response spectra and the UHS. It shows that median response spectrum is higher than 90% of the UHS across a period range of 0.05 s to 1.66 s which covers up to two times the longest fundamental period of structures. A complete incremental dynamic analysis (IDA) [9] was carried out. The suite of ground motions was scaled up by increments of 0.1 g until the collapse of each archetype. The failure criteria was defined as the deformation demand on an element that exceeded its deformation capacity or when the tangent slope of the IDA curve equals 20% of the initial IDA slope [33]. To increase the accuracy of recorded collapse capacity, a linear search between the highest non-collapsing and lowest collapsing point with an acceptable tolerance of 0.01g was performed.

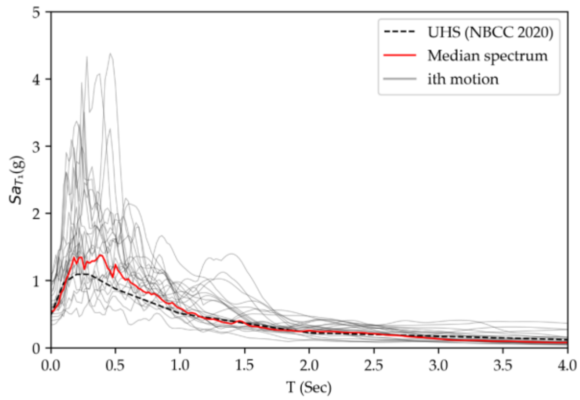

Figure 4. Comparison of response spectra of the FEMA P695 far-field ground motions and uniform hazard spectrum of Vancouver.

PERFORMANCE EVALUATION OF ARCHETYPES

Using the performance evaluation procedure outlined in CCMC [15] guideline, iteration procedures were carried out to evaluate the trial modification factors until satisfied values were achieved. This paper focuses on the final accepted factors which are R_d = 2 for case B, C and E and R_d = 2.5 for case D. Case A demonstrated satisfactory collapse capacities when designed with R_d = 3. At the design intensity (100% UHS), the NBCC sets a Maximum Inter-Storey Drift ratio (MISD) limit of 2.5%. Figure 5 illustrates the median value of MISD under 22 ground motion records until collapse.

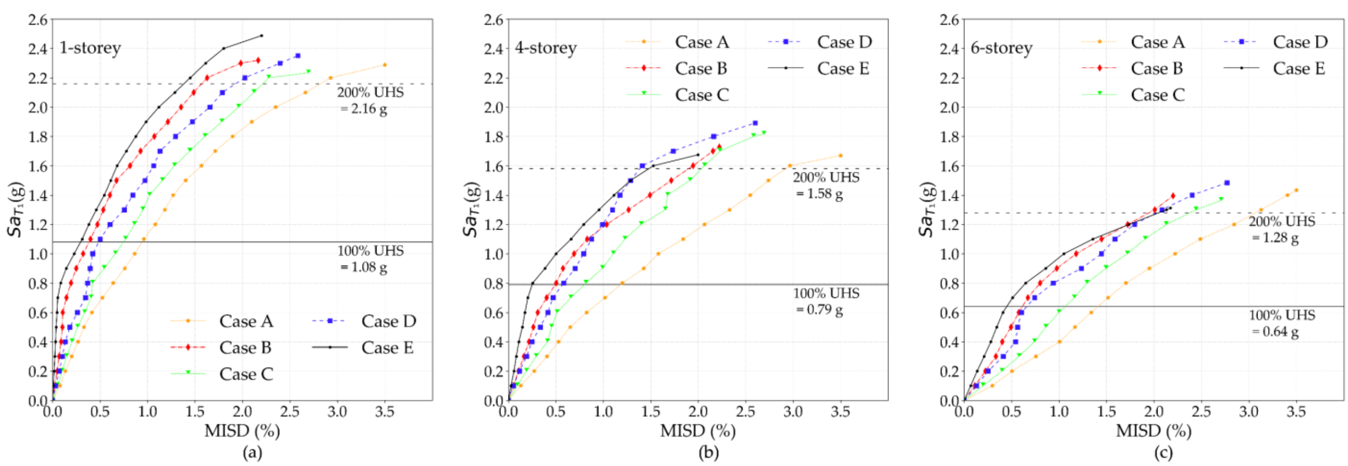

Figure 5. Median IDA curves: (a) single-storey; (b) 4-storey; and (c) 6-storey archetypes

The results clearly indicate that MISD of all archetypes is below 2.5% at the 100% UHS level. Moreover, all archetypes failed at an intensity level more than 200% UHS, which satisfy the second requirement of the CCMC guideline. The collapse MISD ranges between 2% to 3.5%. The high-capacity hold-downs used in case E (pure CLT core) were able to withstand high axial forces and provide the necessary energy dissipation. Among the hybrid cases designed with R_dR_o = 2.0 × 1.5, the MISDs at design intensity (100% UHS) of case B (hybrid structures with 45° STS) were 53%, 40% and 44% less than those of case C (hybrid structures with 90° STS) in 1-storey,4storey and 6-storey archetypes, respectively. Despite of lower deign force (R_dR_o = 2.5 ×1.5), Cased D (hybrid structures with mixed-angle STS) experienced design level drifts that were 33%, 19% and 26% less than case C in 1-storey,4storey and 6-storey archetypes, respectively.

These findings demonstrate that the use of 45° STS connections is considered the most effective means of controlling lateral displacement, but if one aims to both minimize displacement and design with a higher R_dR_o factor, employing mixed-angle STS connections is a preferable option. Nevertheless, it is worth noting that while mixed-angle STS connections can achieve both objectives. Its displacement reduction capability may not be as efficient as that of 45° STS connections at design level intensities but provides a superior seismic safety margin at near-collapse state.

Case A archetypes (pure LWF construction) collapsed at MISD of 3.5% due to LWF walls reaching their ultimate deformation at the top storey. Collapse of Case B, C and E occurred because of hold-down failure, while only archetypes of case D collapsed as a result of connection failure at the top storey. The maximum value of median shear connector displacement at collapse of structures among all archetypes was 11.7 mm and 45° STS connections were deformed up to 2.3 mm in all cases at near-collapse limit state. The adequacy of capacity design procedure was proved since shear connectors and 45° STS connections sustained small deformations. Figure 6 presents the elements' response of case B, C and D under Loma Prieta (Capitola station) in the 6-storey archetypes. Only 1st, 4th and the 6th storeys of the 6-storey are shown because these storeys had more significant contrast in response compared to 2nd, 3rd and the 5th storeys. The connection hysteresis loops indicate that with increase in building height the force demand on connections increases. The element failures that signaled collapse are pointed out in Figure 6 (c, g and j).

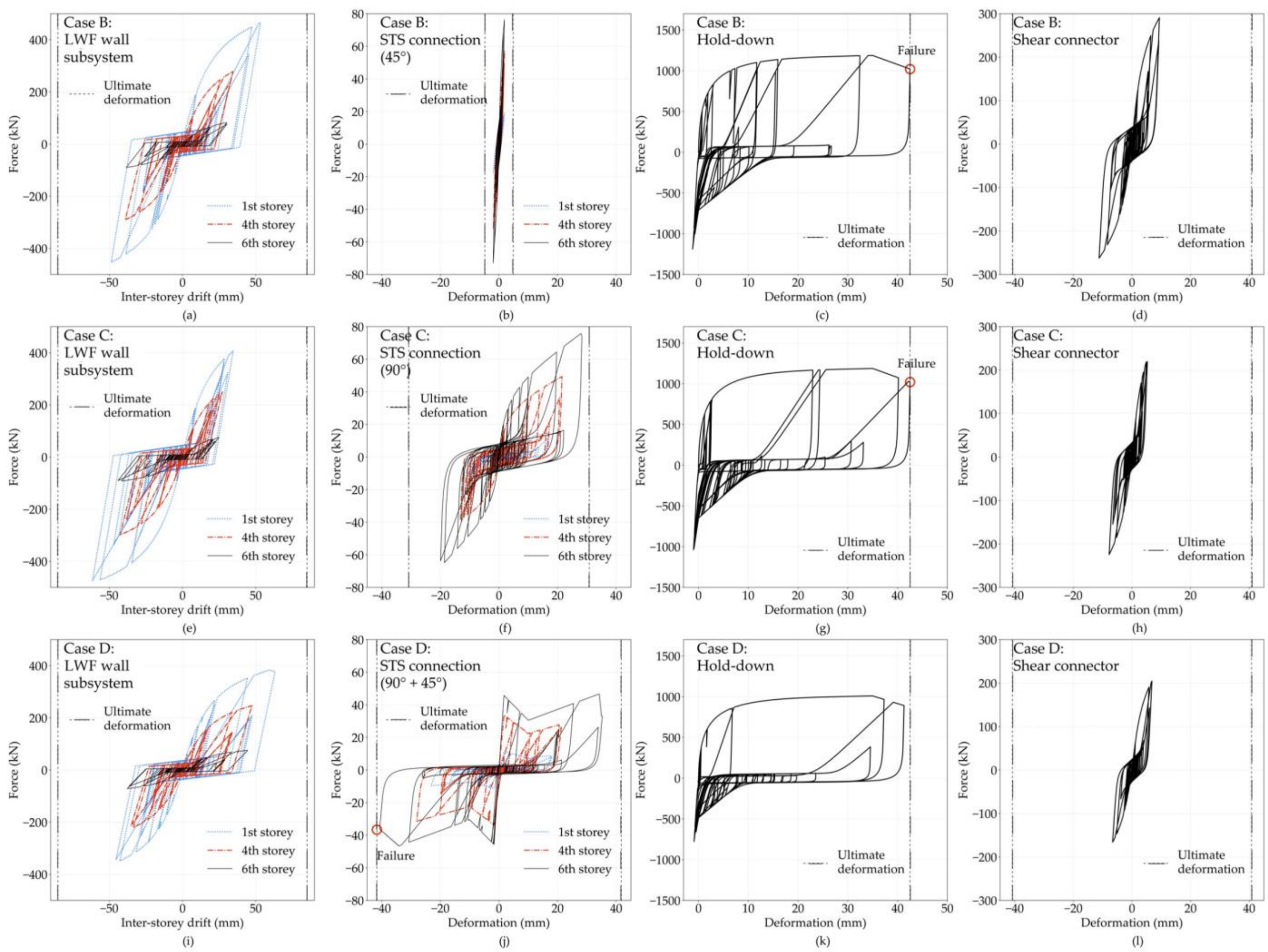

Figure 6. Element force-deformation hysteresis under Loma Prieta at collapse (6-storey archetypes): (a-d) Case B; (e-h) Case C; and (i-j) Case D.

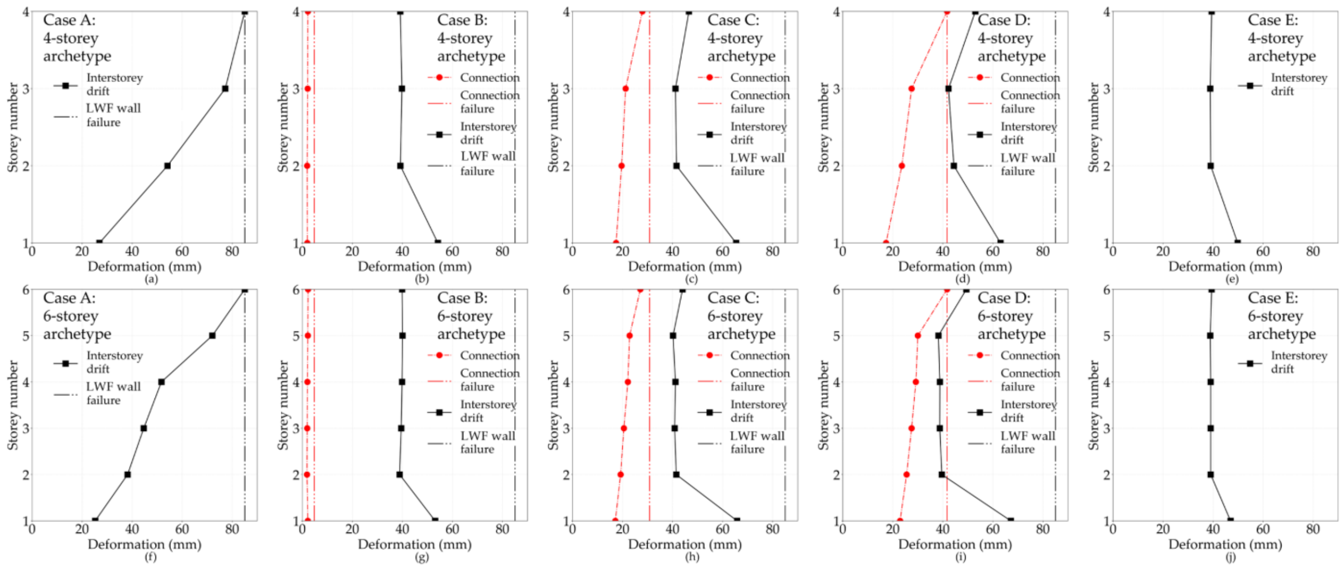

Figure 7. Median interstorey drift and connection deformation at collapse: (a-e) 4-storey archetypes;and (f-j) 6-storey archetypes

The median interstorey drift and connection deformation of multistorey archetypes at collapse are shown in Figure 7 and listed in Table 7. As depicted in Figure 7 (a) and (f), case A exhibited an increasing trend in interstorey drift as the height of the building increased. The connection in the top storey sustained the most damage compared to lower storeys, to the extent that it was the primary cause of collapse in case D. It is worth noting that under 6 out of 22 ground motions dynamic instability occurred before element failure in case D of one-storey structures. In cases with a CLT core, the interstorey drifts were approximately equal in the middle storeys (2nd and 3rd storeys in 4-storey and 2nd to 5th storeys in 6-storey structures) but the first storey significantly experienced higher drifts.

Table 7. Median interstorey drift and connection deformation at collapse

	A1	**B1**				**C1**				**D1**				**E1**	
Storey	LWF	LWF	STS	HD	SC	LWF	STS	HD	SC	LWF	STS	HD	SC	HD	SC
1	85.0	52.8	2.3	42.5	10.7	65.3	25.4	36.5	6.7	63.5	29.3	30.2	5.9	42.5	11.7
	A4	**B4**				**C4**				**D4**				**E4**	
Storey	LWF	LWF	STS	HD	SC	LWF	STS	HD	SC	LWF	STS	HD	SC	HD	SC
4	85.0	39.2	2.2	-	-	46.5	27.9	-	-	53.0	41.7	-	-	-	-
3	77.2	39.8	2.1	-	-	41.1	21.2	-	-	42.3	27.5	-	-	-	-
2	54.1	39.2	1.9	-	-	41.6	19.6	-	-	44.4	23.6	-	-	-	-
1	26.9	54.2	2.0	42.5	11.7	65.4	17.4	42.5	7.6	63.0	17.3	40.9	6.9	42.5	11.2
	A6	**B6**				**C6**				**D6**				**E6**	
Storey	LWF	LWF	STS	HD	SC	LWF	STS	HD	SC	LWF	STS	HD	SC	HD	SC
6	85.0	40.4	2.3	-	-	44.0	27.1	-	-	49.4	41.7	-	-	-	-
5	72.0	40.0	2.2	-	-	40.2	22.8	-	-	38.2	30.0	-	-	-	-
4	51.7	39.9	2.0	-	-	41.1	22.1	-	-	38.8	29.2	-	-	-	-
3	44.6	39.5	1.9	-	-	40.8	20.5	-	-	38.8	27.5	-	-	-	-
2	38.2	38.9	1.9	-	-	41.4	19.2	-	-	39.6	25.5	-	-	-	-
1	25.2	53.1	2.2	42.5	11.6	65.8	17.1	42.5	8.3	67.2	22.9	39.9	6.1	42.5	7.6

CONCLUSION

The focus of the study was to evaluate the effect of ductility of inter-system connections on the R_dR_o of a hybrid LWF/CLT structure connected with self-tapping screws. Three types of connections with 45°, 90°, and mixed angles (45° + 90°) STSs were considered in the archetype development. One-, four- and six-storey structures with pure LWF wall (case A), hybrid buildings with 45° STS (case B), 90° STS (case C) and mixed-angle STS (case D) connections along with a pure CLT core wall system were investigated. The added energy dissipation was expected through yielding of connections in horizontal shear of LWF floor and CLT core wall. The results reported in this work are limited to accepted R_dR_o and the assumed half-half design resisting ratio of the two sub-systems. The following conclusions from this performance analysis are summarized:

- R_d = 2 and R_o = 1.5 were acceptable for hybrid structures with 45° STS and 90° STS connections (cases B, C) and pure CLT structures (Case E). The application of 45° STS and 90° STS connections did not improve the collapse capacity of the hybrid building to make higher R_d values suitable for design. Hybrid system with mixed-angle STS connections (Case D) designed with R_d = 2.5 and R_o = 1.5 satisfied the CCMC requirements. The trial R_d values were limited to 2, 2.5, and 3 (i.e., from 2 to 3 with 0.5 interval) during evaluation.
- Hybrid systems with CLT core wall and LWF system had lower interstorey drift compared to the pure LWF systems. Combining the two systems increased the lateral stiffness of the light-frame wood system, which addresses the rigidity issues often associated with LWF construction.
- The addition of CLT core to the LWF construction led to a more uniform drift distribution in LWF walls. Therefore, it can prevent concentration of damage and extreme responses in one storey.
- The displacement demands on STS connections are larger on higher storeys compared to lower storeys. Therefore, it is important to exercise greater care when designing connections on higher storeys.

Overall, the present study highlights the efficiency of hybrid LWF/CLT structures, particularly when using mixed-angle STS connections to link the two subsystems. This approach effectively addresses the rigidity issues often associated with LWF construction while also enabling a less conservative design. Designers could use a higher R_dR_o when designing the hybrid structure with mixed-angle STS connections compared to the R_dR_o of CLT structures. This approach can lead to more innovative and optimized designs. The current study provided fundamental insights for design of hybrid LWF/CLT structures. However, the study did not consider archetypes with varying level of design resistance ratio between the two subsystems. The connection efficiency may be affected by the resistance ratio of the sub-systems which imposes limitations on generality of results.

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