COLLAPSE RISK OF COUPLED RC-TIMBER HYBRID BUILDING WITH CONSIDERATION OF 2015 NBCC SEISMIC HAZARD

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Abstract

The rapid growth of urban population and associated environmental concerns are challenging city planners/ developers to consider sustainable building systems. The use of wood in structural applications in Canada is promoted by the wood industry and provincial governments. Several provinces, including British Columbia, have adopted a “Wood First Initiative” in order to create the “Culture of Wood Use.” This initiative requires provincially funded projects to use wood as the primary construction material and helps innovative design solutions to bring timber buildings to new heights. This also provides a unique opportunity for designers to take advantage of innovative designs (e.g., hybrid structures) and alternative solutions to overcome the height restriction required by the building code. Part of this initiative is the recently completed project by the authors of this paper, which was funded by the British Columbia Forestry Innovation Investment’s (FII) “Wood First Program.” The main goal of the research project was developing wind and seismic design guidelines for archetype low-, mid-, and high-rise mass-timber buildings. Results related the development of seismic force modification factors, are summarized in this paper.

As compared to the traditional stick timber-frame structures, a cross-laminated timber (CLT) wall is a composite structural assembly with high strength and stiffness properties. In this study, the beneficial properties of this wall were utilized in designing a 20-story coupled CLT-reinforced concrete (RC) hybrid system building, denoted as 20S-C. The structural form of the case study concrete jointed tall mass-timber, first proposed by Skidmore, Owings & Merrill (SOM), is considered. The current seismic design code of Canada uses a force-based design (FBD) procedure. The FBD requires ductility (R_d) and over-strength (R_o) seismic force modification factors to obtain the lateral design loads. For the proposed hybrid system, however, these factors are not available in the national building code of Canada (NBC), and in this paper, they are analytically derived following the FEMA P695 collapse risk procedure. The specific objectives of this research are to develop the R_o and R_d factors for archetype tall mass-timber hybrid buildings and implement the FBD guidelines to design the case study 20S-C following capacity-based design principles.

To study the implication of system ductility on the collapse risk, for R_o = 1.5, three levels of R_d = 2, 3, and 4 were considered. Numerical modeling of the 20S-C was carried out in the Open System for Earthquake Engineering Simulation (OpenSees) finite element framework. The OpenSees model validation entails modeling elements and connections of the system, also calibrating with experimental results where possible, formulating the numerical model for the CLT wall system, and assembling these components to achieve the desired system-level property. Using static pushover to dynamic analysis, suitability of the three R_d factors were investigated. In developing seismic fragility models of a building via nonlinear dynamic analysis, suitable set of ground motion (GM) records that reflect seismic environments of the region. Hence, a suite of GMs, with the seismicity of Vancouver was selected in congruence with 2015 NBC. For the acceptable R_d, using incremental dynamic analysis, the collapse fragility and risk were computed, and suitability of the design factors verified.

Keywords: Hybrid mass-timber building, CLT, seismic risk, coupled system

1. Introduction

The rapid growth of urban population and associated environmental concerns are challenging city planners/developers to consider sustainable building systems. Timber is such a sustainable material [1, 2], and in Canada, the use of wood in structural applications is promoted by the wood industry and provincial governments. Several provinces, including British Columbia, have adopted a "Wood First Initiative" in order to create a "Culture of Wood Use." The 2015 National Building Code Canada (NBC) limits the height of stick frame construction to 6 storeys. The seismic performance of mid-rise timber structures is deemed to be acceptable, as observed in full-scale shake table tests [3]. The evolution of the Canadian timber building code height limit is shown in Fig. 1.

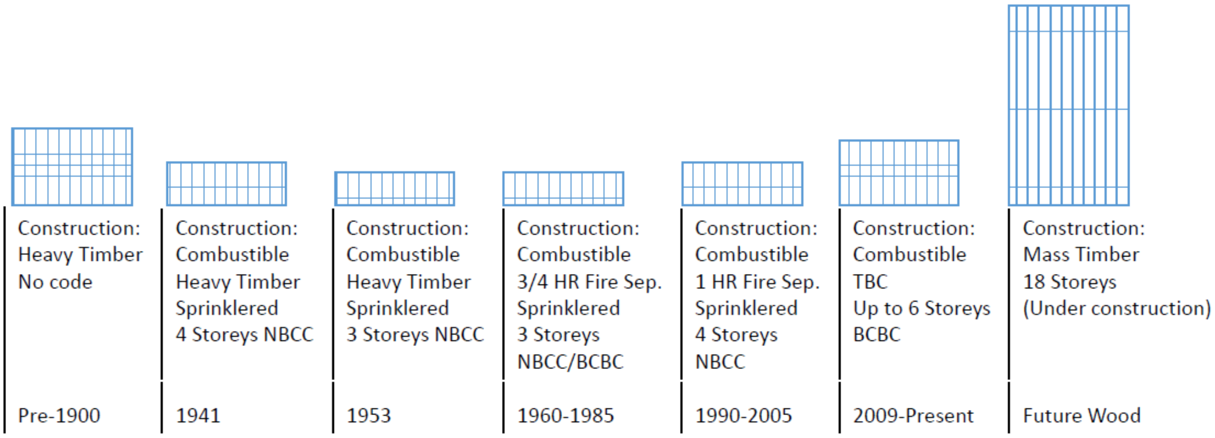


Fig. 1 – Evolution of building height (Green and Karsh [4])

With the recent introduction of large-scale engineered cross-laminated timber (CLT) panels and glulam timber, mid- and high-rise timber buildings became viable option [1, 2, 5, 6]. The current NBC uses a force-based design (FBD) procedure [7], where ductility (R_d) and over-strength (R_o) seismic force modification factors are used to obtain the lateral design loads. The CSA 086 [8, 9] has adopted an R_d = 2 and R_o = 1.5 for platform-type construction using CLT not exceeding 30 m in height. Similar seismic force modification factors were proposed by Pei *et al*. [10]. Building heights up to 42 storeys, for example, can be designed with consideration of hybrid-based timber buildings [11]. However, tall-timber buildings are lighter and more flexible and can potentially be vulnerable to wind loads [12].

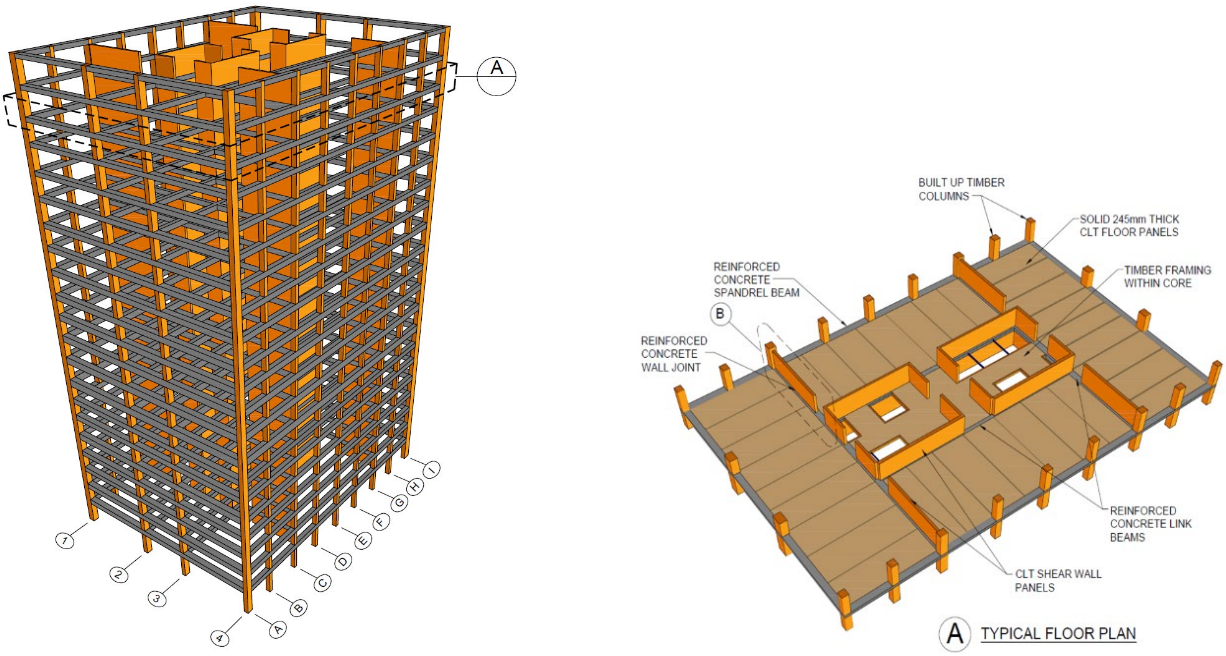
The "Wood First Initiative" requires provincially funded projects to use wood as the primary construction material and promotes innovative design solutions (e.g., hybrid structures) to bring timber buildings to new heights [e.g., 13, 14]. This also provides a unique opportunity for designers to take advantage of innovative designs and alternative solutions to overcome the height restriction required by the building code [e.g., 13]. Part of this initiative is the recently completed project by the authors of this paper, which was funded by the British Columbia Forestry Innovation Investment's (FII) "Wood First Program." The main goal of the research project was developing wind and seismic design guidelines for archetype low-, mid-, and high-rise mass-timber buildings [14]. Results of this project [14], which are related the development of seismic force modification factors of 20 storey building, are summarized in this paper.

In this study, the structural form of a twenty-storey concrete coupled tall mass-timber (20S-C) building proposed by SOM [15] is considered. In this project, the R_o and R_d factors for the 20S-C were developed for the FBD guidelines and detailed using capacity-based design principles. For the hybrid system, however, these factors are not available in NBC [7] and are derived analytically following FEMA P695 [16] collapse risk procedure [e.g., 13, 14]. The FEMA P695 procedure entails consideration of different force modification factors and validate the acceptability through collapse risk. For the 20S-C, trial values of R_o = 1.5 and R_d = 2, 3, and 4 were initially considered and validated. The buildings were designed with the seismicity of Vancouver, British Columbia and unique site-specific hazards are considered. Crustal, inslab, and interface earthquakes contribute most significantly to site-specific seismic hazards at sites in south-western British Columbia [17] and ground motions are selected [18].

2. Design and numerical modeling of the study hybrid mass-timber building

2.1 Building design consideration and layout

A perspective view of the 20-storey coupled mass-timber building (20S-C) and floor plan (dimension of 28.7 m × 40.7 m) are shown in Fig. 2a and Fig. 2b, respectively. The CLT floor panels are designed as a one-way slab system spanning between the central wall systems and periphery framings. Further, the concrete joints over the central walls and the spandrel beams around the periphery are designed to provide fixed support to the CLT floor system. With these assumptions, the CLT floors are designed with 245 mm thickness. With 3.65 m equal storey heights, the total height of the structure is 73 m. Following 2015 NBC, the office building is designed with a live load of 2.4 kN/m^{2,} and 2.8 kN/m^2 superimposed additional load to account the weight of floor finishes and partition load. The building is assumed to be located in Vancouver, BC and the 2015 seismic hazard is considered for design purposes. The site Soil Class is assumed to be Class C (very dense soil or soft rock with average shear wave velocity, V_{s30} = 450 m/s). Snow load is neglected in the design.



(a) (b)

Fig. 2 – RC-timber hybrid building, a) 3D perspective, and b) typical floor layout

2.1 Modeling of CLT shear core walls

A schematic of the RC beam and CLT wall assembly is shown in Fig. 3. A 3-ply-panel (minimum required) CLT infill is used. With 33 mm laminae, overall panel thickness is 99 mm, and the CLT panel is modeled as a linear elastic shell element [e.g., 19, 13, 14]. In the finite element implementation, the 99 mm thick shell section is a significant simplification of the behavior of the panels; however, this result is acceptable as the energy dissipation is controlled by the connection. Similar simplifications were reported in other studies for CLT panel behavior [20]. The numerical model of the walls is accomplished by utilizing four noded quad shell elements. These shell elements account for both in-plane and out-of-plane responses in a 3D modeling domain. For the material model of the CLT wall, the elastic orthotropic multi-axial material of OpenSees is utilized, with effective Young's modulus (9652 MPa) and Poisson's ratio (0.44).

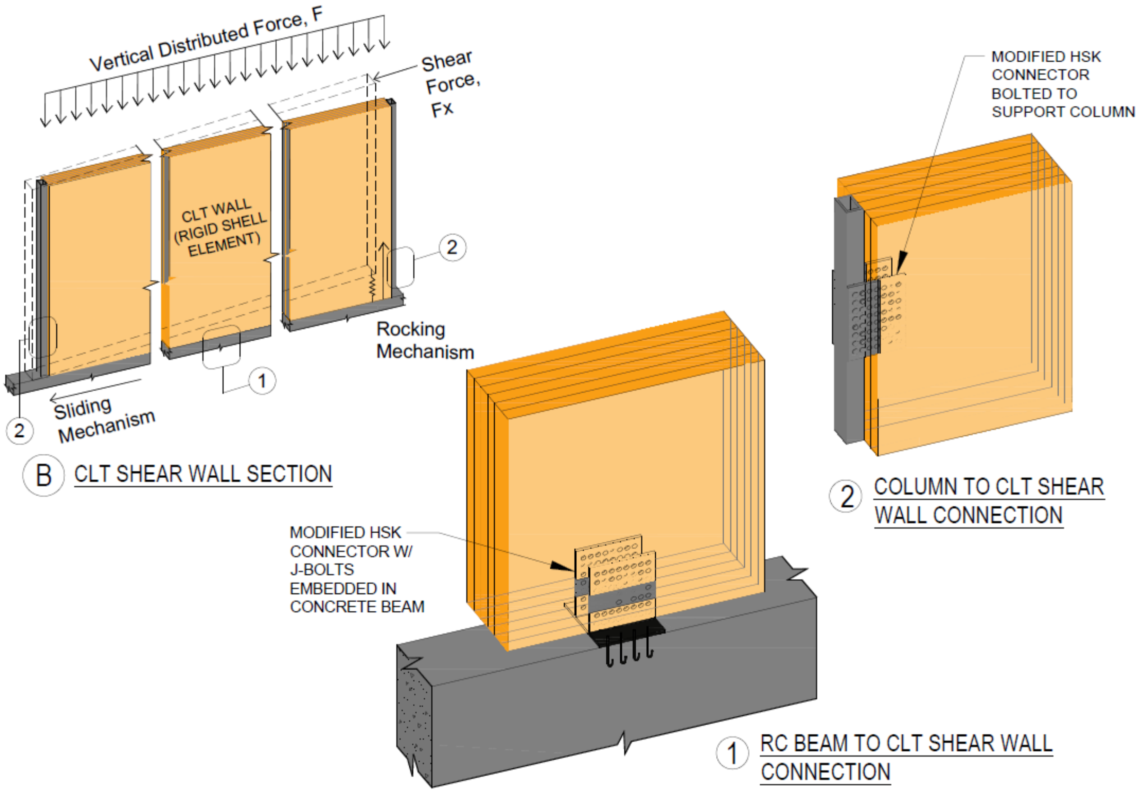


Fig. 3 – CLT shear wall assembly and connection detailing

2.2 Modeling of timber connections and RC coupling beams

Modified HSK connection [21] has high strength and ductility that makes it suitable for use in high-rise mass-timber and hybrid buildings. The modified HSK connection is used both for the hold-downs and the shear connectors (Fig. 3). For the uncoupled system, the HSK connections are the primary inelastic elements participating in the rocking mechanism of the lateral system, whereas for the coupled system, the modified HSK connections respond as secondary fuse elements, and they are expected to yield after the reinforced concrete link-beams have yielded.

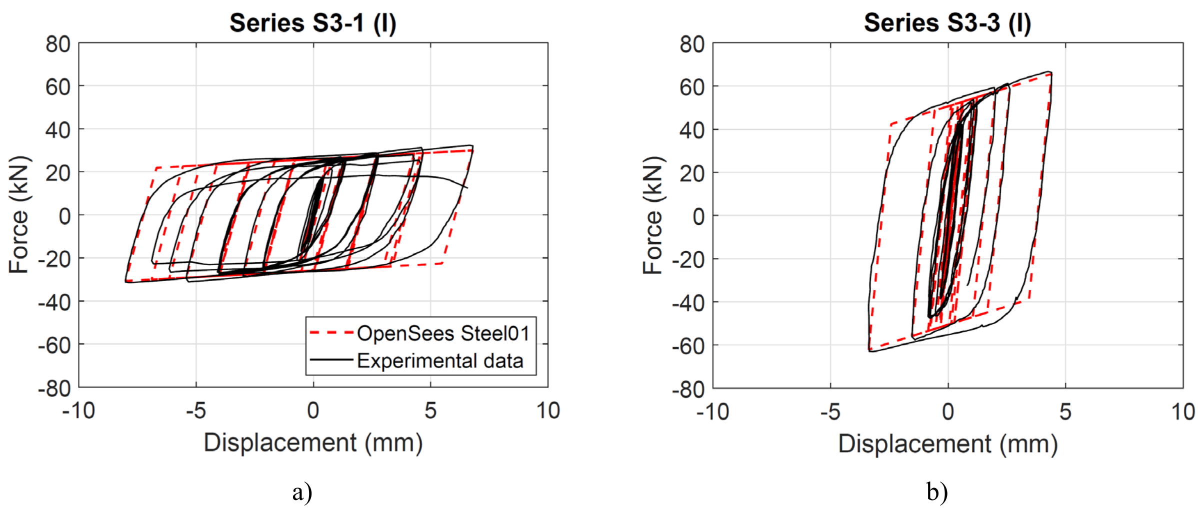
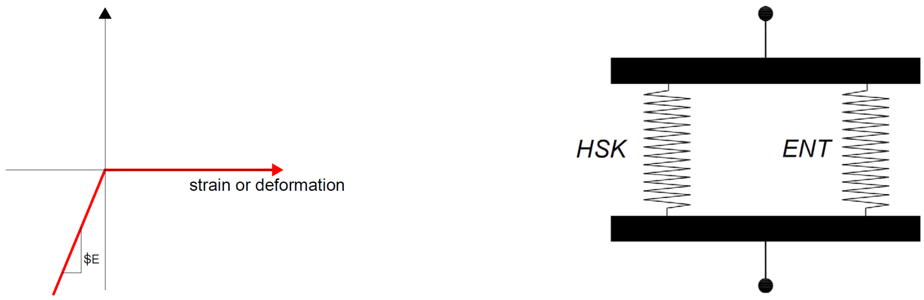


Fig. 4 – Calibrated modified HSK connection: a) with 11 shear links and 2 taped rows; and b) with 23 shear links and 1 taped row

The experimental hysteresis curve results reported in Zhang [21] are used to calibrate the OpenSees's *Steel02 uniaxial material* (Giuffre-Menegotto-PintoModel with Isotropic Strain Hardening). The modified HSK hold-downs and shear connectors are modeled as non-degrading structural members, and their ductility capacity is checked from the analysis results, based on the 40% ultimate displacement values [22]. Fig. 4 shows the experimental and calibrated hysteresis curve. The contact between the CLT wall and the base (ground or RC beam) is accounted for by introducing an elastic no-tension (ENT) element spring in OpenSees (Fig. 5).



a) b)

Fig. 5 – Hold-down assembly models: a) Elastic-no-Tension (ENT) uniaxial material model, and b) parallel formulation of the HSK and ENT uniaxial material models

The reinforced concrete link beams that couple the CLT walls are the main fuse elements in the coupled-system, and they are the first members to yield in the case of a strong seismic event. Hence, in the present study, they are designed as the weakest elements, providing most of the building's ductility, with due consideration given in modeling their inelastic response. The coupling RC beam is shown in Fig. 6. A lamped plasticity model, using the Modified Ibarra-Medina-Krawinkler peak-oriented model hysteretic model [23] (Fig. 7), is utilized.

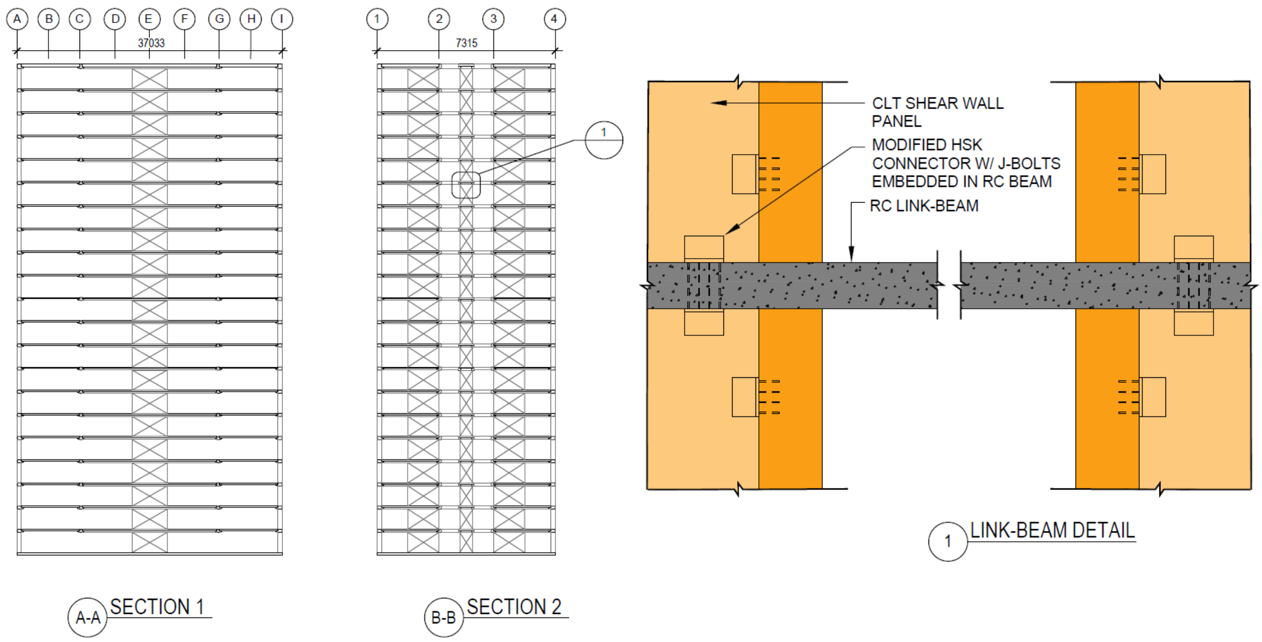


Fig. 6 – RC coupling beam detail

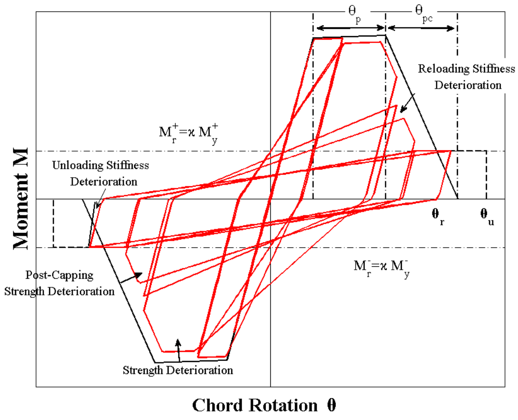


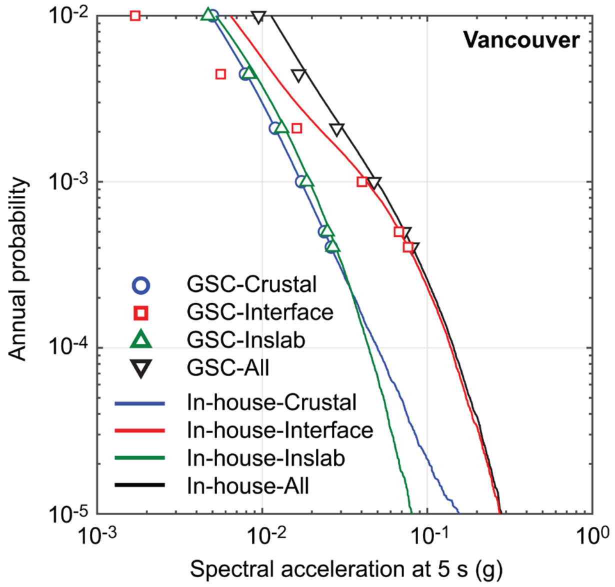
Fig. 7 – Modified Ibarra-Medina-Krawinkler peak-oriented model (http://opensees.berkeley.edu)

3. Seismic hazard in Vancouver and ground motion selection in congruence with 2015 NBC

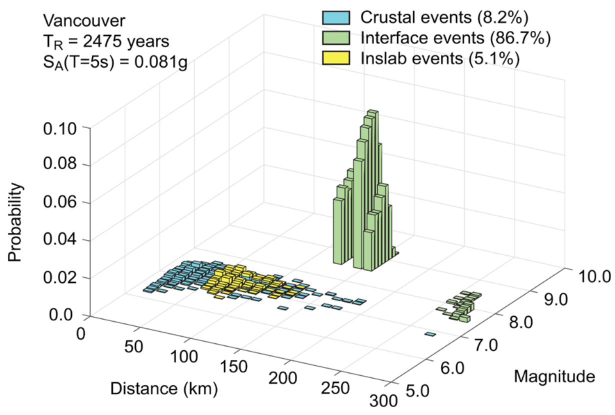
In developing seismic fragility models of a building via nonlinear dynamic analysis, it is important to select a suitable set of GM records that reflect seismic environments of the region. GM records can be selected in various ways. One of the standard approaches is to identify a suite of GM records, response spectra of which match with a target response spectrum at a site of interest. A popular choice in defining target response spectrum is the conditional mean spectrum (CMS) [24], which is a modified uniform hazard spectrum (UHS) that is usually derived for seismic building code purposes (e.g. [17, 25]).

The multiple-event-based CMS ground motion selection requires detailed information on seismic hazard characteristics of contributing sources. The necessary inputs in defining the target CMS and in selecting an appropriate set of records are the UHS corresponding to a return period level (e.g., 1 in 2500 years) and the seismic disaggregation results. The latter is employed to specify plausible earthquake scenarios in terms of mean magnitude and mean distance and to estimate proportions of seismic hazard contributions from different types of earthquakes. Although the site-specific UHS values at numerous locations in Canada are available from Halchuk et al. [17, 25], detailed seismic disaggregation results are generally unavailable.

To obtain detailed information on seismic hazard characteristics of contributing sources, an in-house probabilistic seismic hazard analysis (PSHA) tool is developed based on Monte Carlo simulations [26] by implementing all major components of the national seismic hazard model as described in Halchuk et al. [25]. For the in-house PSHA tool, the length of the synthetic earthquake catalog is set to 50 million years. Fig. 8a compares seismic hazard curves for spectral acceleration at 5 sec $S_A(T=5s)$ (nearest vibration periods to the vibration period of the 20S-C model) based on different earthquake types. At the return period of 2475 years, the differences in the UHS values between the national seismic hazard model and the in-house simulations are less than 2%. Fig. 8b shows both inslab and interface earthquakes have a dominant influence on the calculated seismic hazards for the seismic hazard in Vancouver. To select a suitable set of records based on regional seismic hazard characteristics (Fig. 8), an extensive database of ground motions has been compiled, and the multiple-CMS-based record selection method is implemented. In consideration of significant computational requirements in conducting a series of nonlinear dynamic analysis (as per IDA), the number of records is set to 15. In matching the response spectra of candidate records with the target CMS, the anchor vibration period for record scaling is set to T_A = 3.6 sec. After carrying modal analysis of 10S-U, the first three periods are: T_1 = 3.56, T_2 = 2.78, and T_3 = 2.61 sec. The first-mode vibration period, approximated to T_1 = 3.60, was used as the anchor vibration. Limiting vibration periods, T_{min} = 0.1 sec and T_{max} = 5.0 sec, are considered for the ground motion selection. Details of the developed ground motion database and multiple-CMS-based record selection method can be found in Goda [27]. Fig. 9 compares the response spectra of the selected ground motion records with the target spectrum. Over a wide range of vibration periods from 0.1 sec to 5.0 sec, the match is satisfactory.



(a)



(b)

Fig. 8 – (a) Seismic hazard curves for Vancouver based on different earthquake types, and (b) seismic disaggregation for Vancouver – 2475 years return period and for spectral acceleration at 5 sec ($S_A(T=5s)$)

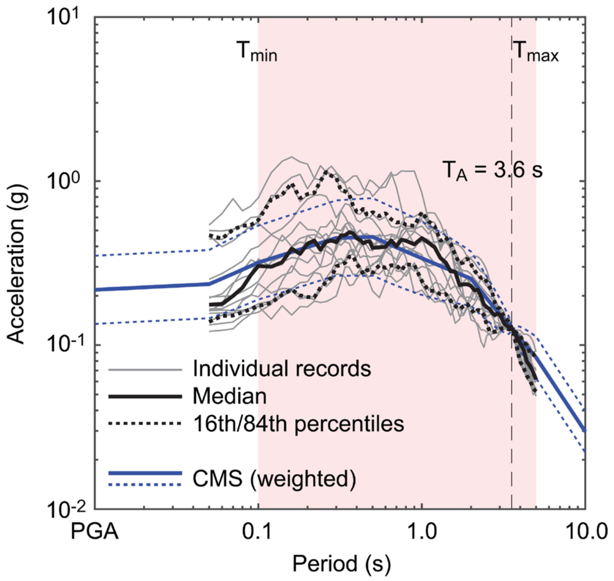


Fig. 9 – Comparison of the response spectra of the selected records with the target response spectrum for $T_A = 3.6$ sec

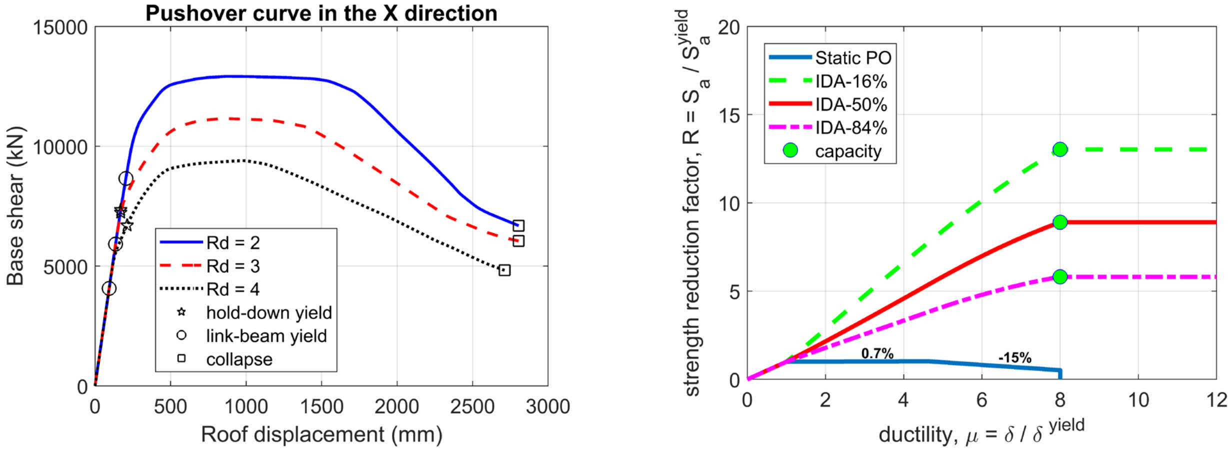
4. Performance assessment

The FEMA P695 [16] determines the acceptability of the modification factors through a collapse risk assessment. The fragility corresponding to the collapse limit state can be obtained using an incremental dynamic analysis [28] that can be computationally intensive to utilize for each trial factor. Thus, in this paper, first, for each building designed with R_d factors of 2, 3, and 4, a static pushover (SPO) analysis was carried out. From the pushover results, preliminary estimation of the collapse risk is obtained using SPO to

IDA (SPO2IDA) procedure [29]. Fig. 10a depicts results of the SPO analysis for the three buildings and Fig. 10b shows the SPO2IDA result for R_d = 2. From the SPO2IDA result (Fig. 10b), the approximate collapse margin ratio ($CMR_{SPO2IDA}$) values are computed as:

$$CMR_{SPO2IDA} = \frac{R_{50\%}}{R_d} \tag{1}$$

where $R_{50\%}$ = median collapse curve from the SPO2IDA analysis; and R_d = ductility factor used in the design. Preliminary results of $CMR_{SPO2IDA}$ values obtained from the SPO2IDA results are summarized in Table 1. The acceptable CMR in FEMA P695 is 2.61 [see Tesfamariam et al. [14] for further detail]. From the results summarized in Table 1, both R_d = 2 and 3 are acceptable factors.



(a) (b)

Fig. 10 – (a) Pushover analysis for the 20-storey coupled (20S-C) building: R_d = 2, 3 and 4, (b) IDA curves generated using SPO2IDA analysis

Table 1: Preliminary CMR values for 20-storey coupled building based on SPO2IDA and IDA

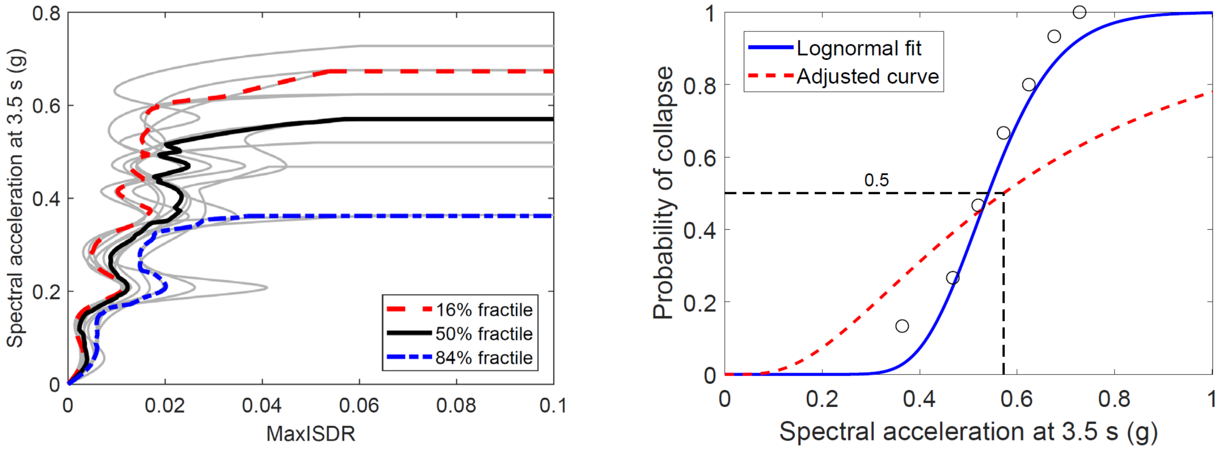
Building ID	R_d	$R_{50\%}$	$CMR_{SPO2IDA}$	$ACMR_{IDA}$
20S-C	2	8.9	4.45	4.6
	3	9.9	3.29	
	4	10.0	2.51	

Furthermore, to verify the acceptability of the trial R_d factor, using the 15 GMs selected in the previous section, the IDA is carried out for R_d = 2. In IDA, each GM is scaled up until sway mode collapse is achieved (maximum inter-storey drift ratio, MaxISDR = 5%). Fig. 11a depicts the IDA curves for the 20S-C building (R_d = 2). The horizontal axis represents the MaxISDR and vertical axis is spectral acceleration at the fundamental period of the building S_A(T=3.6).

The fragility curves reflect the probability of collapse of the hybrid buildings. These curves are cumulative distribution functions (CDF) developed by fitting a lognormal distribution through collapse intensity values for all GMs. The collapse S_A(T=3.6) values shown in Fig. 11a are fitted with a log normal distribution to obtain collapse fragility (Fig. 11b). The mean and standard deviation are obtained to be 0.54 and 0.21, respectively. From the IDA collapse fragility curves, the CMR is computed as:

$$CMR = \frac{\hat{S}_{CT}}{S_{MT}} \tag{2}$$

where $\hat{S}_{CT}$ = median collapse capacity, and S_{MT} = spectral acceleration value from the 2% in 50 years hazard spectrum at the fundamental period of the archetype structure.



(a) (b)

Fig. 11 – Results of nonlinear time history analysis: a) incremental dynamic analysis (IDA), b) collapse fragility for 20S-C building and $R_d = 2$

The fragility shown in Fig. 11b, is modified with the total collapse uncertainty (β_{TOT}) [14, 16] that accounts for: record-to-record (β_{RTR}=0.4), design requirement (β_{DR}=0.35), modeling (β_{MDL}=0.35), and test data (β_{TD}=0.35). Thus, the β_{TOT} is used to modify the original fragility curve as shown in Fig. 11b and is computed as:

$$\beta_{TOT} = \sqrt{\beta_{RTR}^2 + \beta_{DR}^2 + \beta_{TD}^2 + \beta_{MDL}^2} \tag{3}$$

Finally, based on these selected values, the total uncertainty was calculated using Eq. (3) to be 0.726 (β_{TOT} ~ 0.75) and the adjusted collapse fragility is plotted (Fig. 11b). The ACMR is computed to be 4.6 (Table 1). This value indeed is in good agreement with the SPO2IDA results summarized in Table 1.

FEMA P695 [16] proposed acceptability criteria to verify the adequacy of initially assumed force reduction factors based on $ACMR_{10\%}$ and $ACMR_{20\%}$. The R_d factors are accepted if the calculated ACMR ratios were within the performance group and individually exceeded the values provided in [14]. Accordingly, the proposed R_d factor are accepted if the calculated average ACMR values within the performance group ($ACMR_{10\%}$) exceeds 2.61. Moreover, for individual criteria, the proposed factors were acceptable if the calculated ACMR value ($ACMR_{20\%}$) exceeds 1.88. Since there exist one archetype per performance group, the $ACMR_{10\%}$ = 2.61 was used for each archetype conservatively. Based on this assessment, the $R_d = 2$ is acceptable.

5. Conclusions

In this paper, new seismic force modification factors are developed for concrete jointed tall hybrid mass-timber building. The case study hybrid structure incorporates CLT panels for both building floors and laterally resisting shear-walls. For connecting the CLT walls to the upper and lower storey concrete (spandrel) beams, the modified HSK connection was used for both the hold-downs and the shear connectors. The FEMA P695 [16] was followed in this study to develop the archetype model and to quantify the seismic force modification factors. For the 20-story mass-timber building and $R_o = 1.5$, three ductility factors, i.e., R_d = 2, 3 and 4, were parametrically studied. Based on the results of IDA and collapse risk analysis, $R_d = 2$ deemed to be appropriate for the studied system by providing an ACMR value that is within the permitted range of FEMA-P695. The $R_d = 3$ is also acceptable based on the SPO2IDA result, but further validation is required.

6. Acknowledgments

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