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EFFECTS OF CLT-INFILL WALLS ON THE COLLAPSE BEHAVIOR OF STEEL MOMENT RESISTING FRAMES

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ABSTRACT

Over the past six years, to increase the use of renewable materials in the construction industry, a novel steel-timber hybrid building system was developed and studied at the University of British Columbia and FPInnovations. The hybrid structural system was a steel moment resisting frames (SMRFs) with Cross Laminated Timber (CLT) infill walls. These studies were mainly on developing: novel connection types, new constitutive laws for the CLT walls, and force-based and displacement-based design guidelines. The effect of CLT infills on the collapse risk of the SMRFs was not explicitly investigated, and is the topic of this paper. With consideration of seismicity of Vancouver (Canada)and using the 2010 National Building Code of Canada (NBCC) force based design guideline, 3- and 6-storey, 3-bay, bare and middle bay CLT-infilled SMRFs, were designed. Nonlinear analytical building models that account for the frame-infill interactions, were developed in the OpenSees finite element tool. L-shaped steel bracket connections were modeled using experimentally calibrated nonlinear two-node-link elements. Moreover, to allow brackets deformation, a small gap was provided at the interface of the steel frame members and CLT infill panels. To assess the collapse behavior and collapse fragility curves, incremental dynamic analysis was performed using 60 ground motion records selected with seismicity of Vancouver. The infill panels have significantly increased the collapse margin ratio, thereby reducing the collapse risk of SMRFs during server earthquake events.

Keywords: Cross Laminated Timber; Hybrid Structures; Incremental Dynamic Analysis; Collapse Fragility Curves

1. INTRODUCTION

Over the past six years, to increase the use of renewable materials in the construction industry, a novel steel-timber hybrid building system was developed and investigated at The University of British Columbia (UBC) and FPInnovations (Dickof 2013, Stiemer et al. 2012a, b). The hybrid system considered the use of steel moment resisting frames (SMRFs) with cross laminated timber (CLT) infill walls (Figure 1). Dickof et al. (2014) have developed preliminary overstrength and ductility factors using nonlinear static pushover analysis. Tesfamariam et al. (2014), through nonlinear time history analysis, showed the contribution of CLT-infill walls in reducing the seismic vulnerability of SMRFs. Despite the physical gap in the interface to isolate the two systems, under peak lateral load, their interaction may create undesirable shear demand on the steel columns. Bezabeh (2014) and Bezabeh et al. (2015) developed and applied a new direct displacement based design procedure by considering CLT-infill walls as structural elements. Moreover, to simplify the routine structural design of this hybrid structure, the over-strength and ductility factors, and corresponding force-based design guideline were developed as per NBCC 2010 (NRC 2010) by UBC and Forestry Innovation Investment (Tesfamariam et al. 2015). In this paper, the study is extended to quantify the effect of CLT infill walls on the collapse behaviour of the SMRFs.

Figure 1: Steel-timber hybrid building: CLT infilled SMRFs

2. SEISMIC DESIGN OF CASE STUDY BUILDINGS

For this study, 3- and 6-storey, 3-bays bare and middle bay CLT-infilled SMRFs office buildings located in Vancouver, Canada were considered. The buildings were regular both in plan and elevation. For all buildings, the bay widths considered were 9 m for the exterior bay and 6 m for the interior bay. A typical storey height was 3.65 m, except for the first storey which was 4.5 m. In the hybrid buildings, connection brackets were spaced at 800 mm with three layers of CLT panel (99 mm thickness). Panel crushing strength was set to 11.5 MPa. All buildings were designed based on equivalent static procedure of NBCC 2010 (NRC 2010) by considering the soil class C design spectra of Vancouver, Canada. For the bare SMRFs, overstrength (R_o) and ductility (R_d) factors were 1.5 and 5, respectively, according to NBCC 2010 (NRC 2010). Whereas, for the hybrid buildings, R_o and R_d factors of 1.5 and 4, respectively, as suggested by Tesfamariam et al. (2015) were used. Tesfamariam et al. (2015) developed R_d and R_o factors for CLT infilled SMRFs by considering the monolithic action of the hybrid building under lateral load. Typical office floor and roof dead and live loads of NBCC 2010 (NRC 2010) were adopted. The steel members were selected and detailed based on CSA S16-09 (CISC 2010) requirement. Tables 1 and 2 summarize the selected steel sections.

Table 1: Designed beam sections

	Storey No.	6-storey bare	6-Storey hybrid	3-Storey bare	3-Storey hybrid
External	6	W310×45	W310×74		
External	5	W310×52	W310×74		
External	4	W310×67	W310×86		
External	3	W310×86	W310×86	W310×52	W310×60
External	2	W310×86	W310×86	W310×52	W310×60
External	1	W310×86	W310×86	W310×45	W310×60
Internal	1	W310×74	W310×79	W310×45	W310×45
Internal	2	W310×74	W310×79	W310×45	W310×45
Internal	3	W310×74	W310×79	W310×33	W310×45
Internal	4	W310×67	W310×79		
Internal	5	W310×52	W310×67		
Internal	6	W310×45	W310×67		

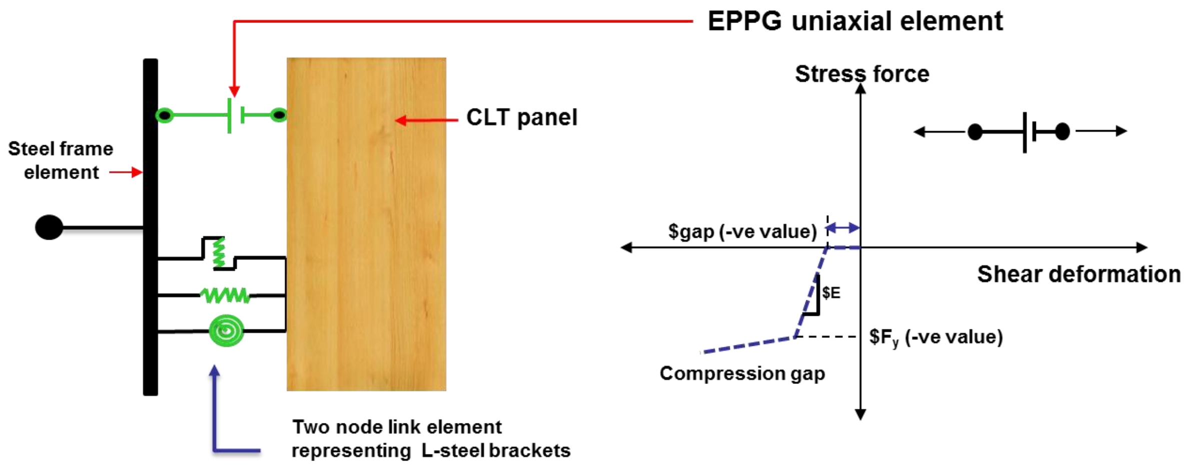
Table 2: Designed column sections

	Storey No.	6-storey bare	6-Storey hybrid	3-Storey bare	3-Storey hybrid
External	6	W310×107	W310×86		
External	5	W310×107	W310×86		
External	4	W310×129	W310×129		
External	3	W310×129	W310×129	W310×74	W310×60
External	2	W310×129	W310×129	W310×74	W310×60
External	1	W310×129	W310×129	W310×79	W310×67
Internal	1	W310×143	W310×129	W310×107	W310×67
Internal	2	W310×143	W310×129	W310×107	W310×60
Internal	3	W310×143	W310×129	W310×107	W310×60
Internal	4	W310×143	W310×129		
Internal	5	W310×129	W310×86		
Internal	6	W310×129	W310×86		

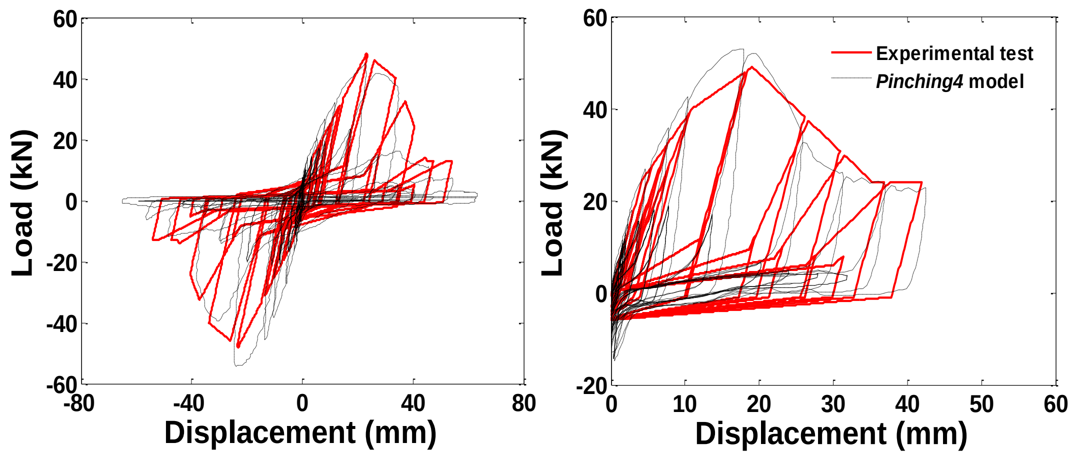
3. FINITE ELEMENT MODEL DEVELOPMENT

Finite element numerical modeling was carried out using Open System for Earthquake Engineering Simulation (OpenSees) finite element program (Mazzoni et al. 2006). First steel frame members were modeled. The nonlinear behavior at the end of these elements was captured by *displacement-based-beam-column-elements*. *Linear-elastic beam-column-elements* were used to model the middle part of steel frame elements. *Modified-Ibarra-Krawinkler-Deterioration-model* (Lignos and Krawinkler 2010) was used as a deterioration model by *bilinear-material* property of OpenSees to capture the spread of inelasticity.

CLT panels were considered as linear-elastic, homogenous and isotropic single layer shell elements with elastic modulus of 9,500 MPa. The in-plane behavior of these elements were modeled using *four-node-quad-elements*. In OpenSees, these elements were characterised by *ndMaterial-Elastic-Isotropic-material* model. The connections at the interface of the steel frame members and CLT infill panels was represented by zero length *two-node-link-element* (Figure 2a). An experimentally calibrated *Pinching4-uniaxial-material model* was used as to represent the axial, shear, and rotational behaviour of these elements (Figure 2b). Additional details of experimental connection tests and *pinching4* model calibration can be found in Tesfamariam et al. (2015). Since this element has zero length, P-Δ effects along the local axis were neglected.



a)



b)

Figure 2: Details of steel bracket connection; a) parallel formulation of *two-node-link-element* and *gap-element* (Tesfamariam et al. 2015); b) Comparison of experimental and OpenSees pinching4 material model (Bezabeh et al. 2015)

The confinement behavior and the physical space between the frame and panel was modelled using the *elastic-perfectly-plastic-gap-uniaxial-material* (EPPG). EPPG is a trilinear hysteretic uniaxial material model which consists of a physical gap (20 mm) with zero stiffness and strength, linear elastic region, and post-yielding plastic region (Mazzoni et al. 2006). In the current case, the compression only gap model was considered to represent the confinement property. Accounting of densification of wood after crushing, the post-yield stiffness of the panel was assigned to be 1% of the elastic panel stiffness. The EPPG gap material and the *two-node-link-element* of bracket connection were combined using the parallel material combination approach as shown in Figure 2a.

4. GROUND MOTIONS

In this paper, the updated seismic hazard model by Atkinson and Goda (2011) was adopted to characterize the seismic hazard in Vancouver. The site condition for probabilistic seismic hazard assessment was set to site class C. Initially modal analysis was performed to calculate the natural periods of each building corresponding to the first three modes. The first mode fundamental period was used for record selection and scaling, whereas the second and third mode periods were used as limiting values to define the range of spectral marching. The record selection was conducted based on a multiple-conditional-mean-spectra (CMS) method (Goda and Atkinson 2011). Using the target CMS, a set of ground motion records was selected by comparing response spectra of candidate records with the target spectra. For each building, the total number of selected records was set to 30 (note: each record has two horizontal components). For example, for the 3-storey hybrid structure, 13, 4, and 13 records were crustal, interface, and inslab earthquakes, respectively. Figure 3 depicts the response spectra of the selected ground motion records for 6-storey hybrid building. The details of ground motion selection, response spectra, and seismic hazard deaggregation for considered hybrid buildings are reported in Tesfamariam et al. (2015).

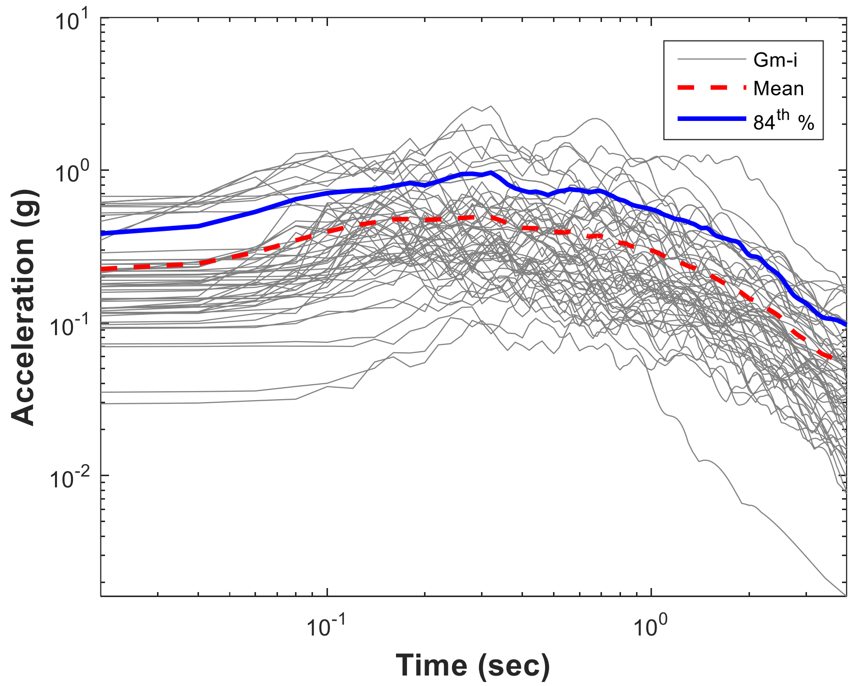


Figure 3: Response spectra of selected ground motion records for 6 storey hybrid building

5. INCREMENTAL DYNAMIC ANAYLSIS

To quantify collapse fragility, incremental dynamic analysis (IDA) (Vamvatsikos and Cornell 2002) was conducted using the 60 ground motion records. In this approach, the intensity of each ground motion is scaled up until the sway mode collapse is achieved. Typically IDA curves are defined using Intensity measure (IM) and corresponding engineering demand parameter (EDP). In this paper, maximum interstorey drift ratio (MISD) and the 5% damped spectral acceleration at the fundamental period ($S_T(T_1)$) were considered as EDP and IM, respectively. As per FEMA P695 (2009) suggestion to check the collapse safety of code based designed buildings, the theoretical fundamental period (T_1) was used for ground motion intensity scaling during the IDA analysis. In this case, as both infilled system and bare system have the same height, their fundamental period as per NBCC 2010 (NRC 2010) is the same. The T_1 for the 3 and 6 storey frames are 0.54 sec and 0.88 sec, respectively. The data from IDA was used to calculate the median collapse intensity (S_{CT}). A conservative collapse criteria was used to define the dynamic sway mode collapse of buildings. Structural hardening was only considered for MISD values less than 10% and the spectral acceleration value corresponding to the dynamic instability was considered as a collapse limit state point. The IDA results are plotted in Figure 4. In Figure 4, each line represents the time history response of the building under single ground motion record. The points on each line show the MISD value corresponding to the intensity level of the ground motion.

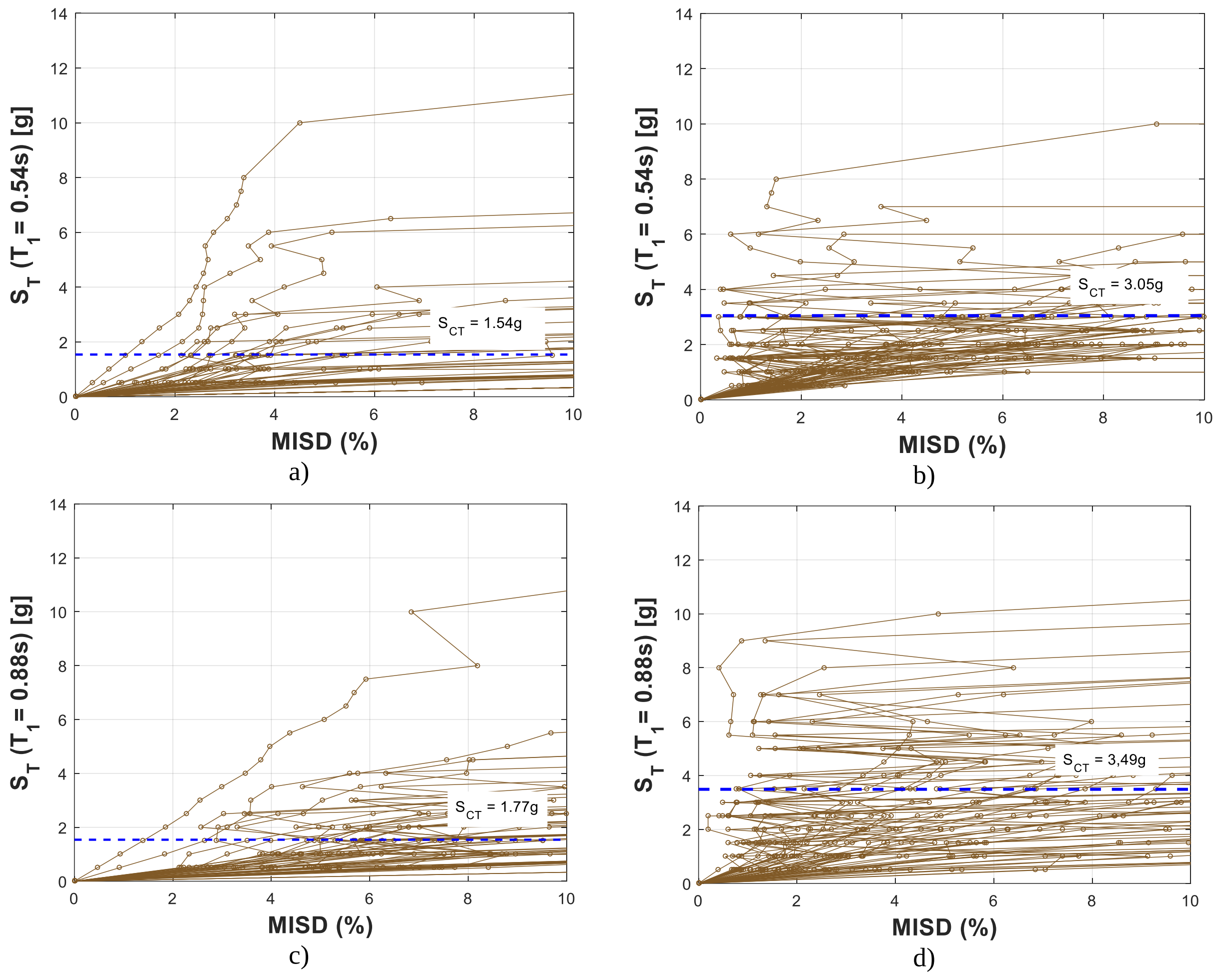


Figure 4: IDA results for 3- and 6-storey buildings a) 3-storey bare frame ; a) 3-storey CLT infilled frame ; a) 6-storey bare frame; a) 6-storey infilled frame

Seismic fragility curves were computed from the IDA results for three EDP values: 2.5%, 5%, and collapse. NBCC 2010 (NRC 2010) and FEMA-356 (2000) represent an extensive damage on SMRFs by EDP of 2.5% and 5%, respectively. These curves reflect the exceedance probability of an EDP when the structure subjected to a given ground motion IM. A fragility function fitting algorithm developed by Baker (2014) was used for this analysis. This algorithm was employed to develop the cumulative distribution functions (CDFs) by fitting a lognormal distribution of IMs at EDP of interest. The lognormal distribution of IMs was defined by median collapse intensity (S_{CT}) and record to record variability (B_{RTR}). Figure 5 shows the drift exceedance and collapse fragility curves for both bare and hybrid buildings. The fragility curves corresponding to an EDP of 2.5 % reflects the probability of exceeding the collapse prevention limit state of NBCC 2010 (NRC 2010). Irrespective of the presence of CLT infill walls, the 2% in 50 years uniform hazard spectral acceleration value of the code-based fundamental period of the building (S_{MT}) for 3- and 6-storey buildings are 0.72g and 0.5g, respectively. At this point it is to be noted that the fundamental periods for S_{MT} computation were calculated by the NBCC 2010 (NRC 2010) equation, which is only a function of height of the building. Considering the collapse damage measure EDP at T_1, for 3-storey bare frame, there is a 16.2% probability of exceedance. Whereas, for the CLT infilled 3-storey hybrid building, the probability of exceeding collapse prevention limit state is 1.2%. In general, significant reduction in the exceedance probability of collapse prevention limit state is obtained by introducing CLT infill walls in 3- and 6-storey steel moment frame structures. Based on static and dynamic analysis, similar results have been reported elsewhere (Tesfamariam et al. 2014 and Dickof et al. 2014).

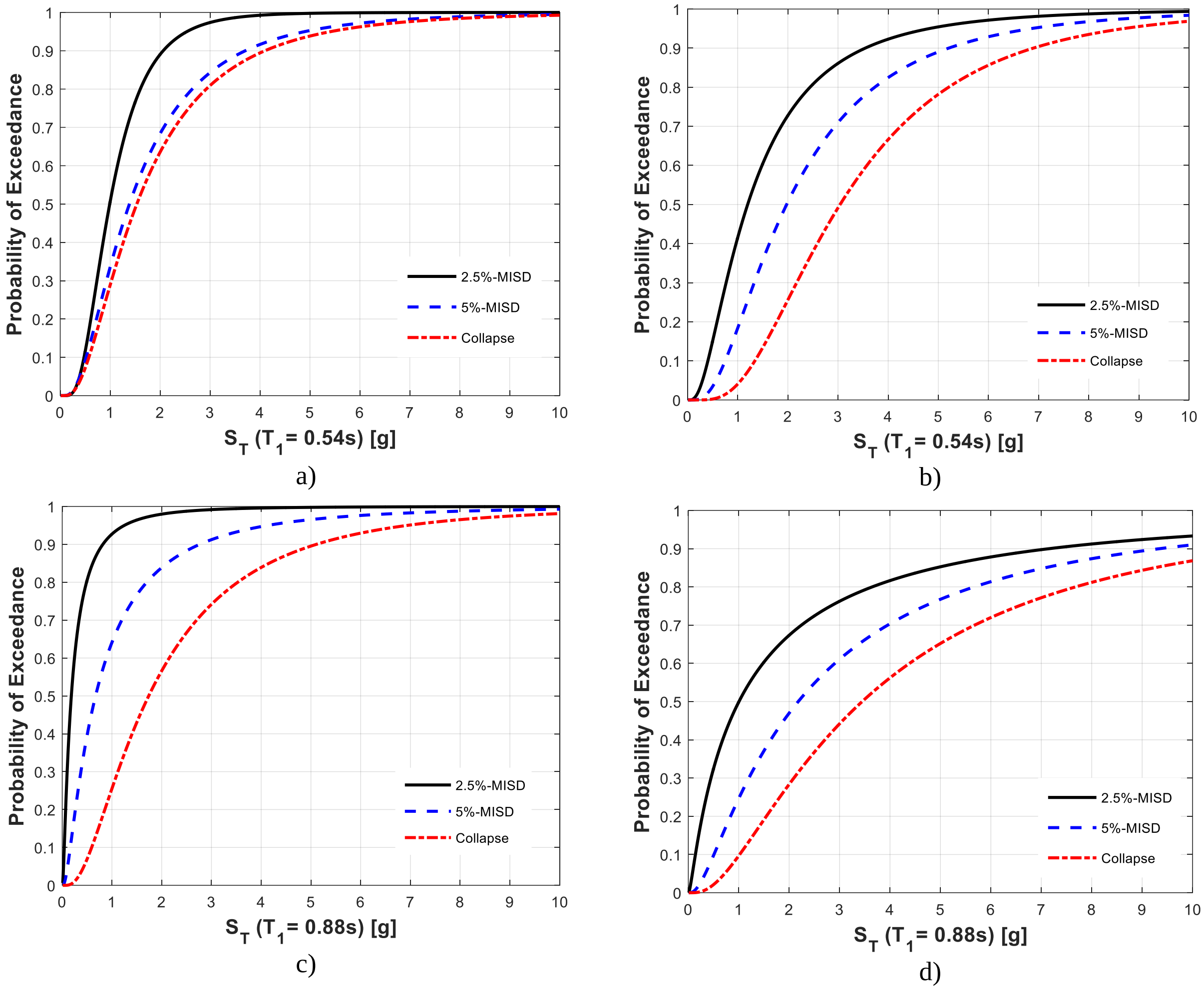


Figure 5: Fragility curves for 3- and 6-storey buildings a) 3-storey bare frame ; b) 3-storey CLT infilled frame ; c) 6-storey bare frame; d) 6-storey infilled frame

FEMA P695 (2009) defines the collapse the safety of seismic force resisting system through collapse margin ratio (CMR), which is a factor to increment S_{MT} to initiate the collapse of the building by half of the ground motion record. Once the median collapse intensity is obtained from the IDA results (e.g. Figures 4 and 5), CMR can be calculated using (FEMA P695, 2009):

[1] $CMR = \frac{S_{CT}}{S_{MT}}$

Table 3 summarizes and compares the calculated CMR values of each building. Generally, irrespective of the height of the building, the CLT infill panels increase the CMR values by enhancing structural stiffness and strength. Due to their larger fundamental period and lower S_{MT} value, of all the considered building types, mid-rise hybrid building shown to have higher collapse safety. The obtained results showed the efficiency of the seismic base shear modification factors proposed by Tesfamariam et al. (2015). Moreover, from the IDA analysis, no premature failures such as a soft storey mechanism and large strength degradation due to panel crushing were seen. Therefore, the ductility and overstrength related factors suggested by Tesfamariam et al. (2015) yield economical and collapse safe buildings.

Table 3: Results of IDA

Building type	No. of storey	Fundamental periods (modal analysis)	Infilled bays	S_{MT} (g)	S_{CT} (g)	CMR	Percent increase in CMR
Low-rise	3	1.59s	bare	0.72	1.54	2.14	49.50%
	3	0.92s	2nd bay	0.72	3.05	4.24	
Mid-rise	6	2.64s	bare	0.5	1.77	3.54	49.40%
	6	1.67s	2nd bay	0.5	3.49	6.98	

6. CONCLUSIONS

In this paper, the effect of CLT infill walls on the collapse safety of bare SMRFs was evaluated. For this purpose, IDA was performed on bare SMRFs and hybrid buildings using the 60 ground motion records. The collapse safety and the exceedance probability of collapse prevention limit state were evaluated using CMR values and seismic fragility curves, respectively. The results showed the benefit of CLT infill panels in enhancing the collapse safety of steel moment resisting frames. For 3 storey frame, by introducing CLT infill walls in SMRFs, the probability of exceeding collapse prevention limit state decreased from 16.2% to 1.2%. Moreover, for 6-storey buildings, the collapse margin ration increased by 49.4%. Of all the analysed buildings, mid-rise hybrid building shows higher collapse safety.In general, significant reduction in the exceedance probability of collapse prevention limit state and sway mode collapse probability is obtained by introducing CLT infill walls in 3- and 6-storey steel moment frame structures. The present study reveals the significance of considering CLT infill walls during the design process to benefit from their contribution to the stiffness, strength, and ductility of the bare steel frames.

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