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Effect of wall curtailments and infill configurations on the seismic performance of CLT shear-wall and Glulam moment resisting frame system

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ABSTRACT

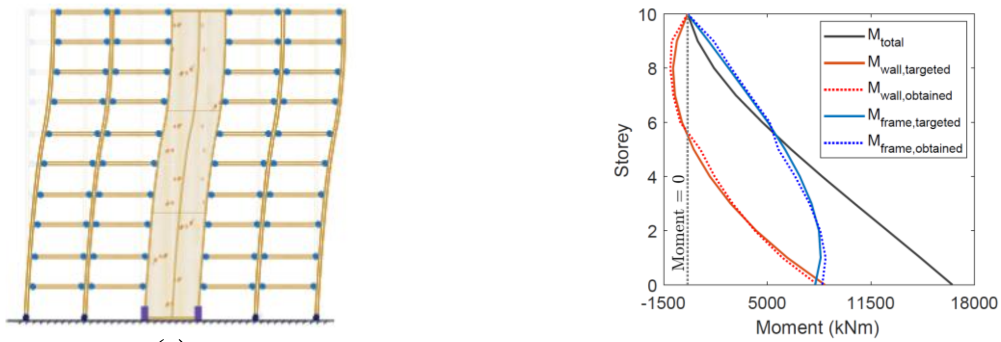
Timber-based building is rising in popularity due to urban densification and the demand for sustainable materials. Over the past two decades, several novel timber products and systems have been developed. A dual timber-based system, CLT shear-wall and Glulam moment resisting frame (CLTW-GMRF), is a recently completed research project prepared for the British Columbia Forestry Innovation Investment Ltd. The proposed system is designed by proportioning the moment contribution of each lateral load resisting systems (LLRSs). The outcomes of this study, however, demonstrated that due to the different mode of deformation of the two LLRSs under seismic forces, an interaction develops and negative-moment is created on the upper part of the wall system. This interaction is significant for tall buildings where their shear-walls are slender. Accordingly, this abstract extends the study on CLTW-GMRF system and investigates the effect of wall curtailments and infills configurations in the general behavior of a typical 10-storey CLTW-GMRF system. Specifically, the change in the bending moment and base-shear distribution of each systems and the beam-column joint moment demands are computed. Moreover, two-dimensional numerical model of the system is developed in OpenSees and the performance of the different systems is examined using 30 ground motion records selected to represent the seismicity of Vancouver - Canada. The impact of the wall curtailments and infill configurations on interstorey and residual drifts, floor acceleration, collapse fragilities and collapse margin ratios are evaluated. It is shown that with the increase in the availability of ductile and high moment-capacity joints, CLTW-GMRF systems with different wall configurations can be a viable alternative for tall mass-timber constructions.

Keywords: Cross-laminated timber, dual system, glulam moment resisting frame, wall curtailments, wall infills.

INTRODUCTION

The increasing need for sustainable, cost-effective, and high-strength materials in modern construction has led to a resurgence in popularity for timber-based construction [1]. Over the past few decades, numerous timber-based structural systems have been proposed in literature (e.g., [2-5]). Despite their material efficiency and architectural openness, moment-resisting timber frames have received less attention than timber-based shear-walls. In these systems, semi-rigid beam-to-column connections can ensure resilience and energy dissipation. Bolted slotted-in steel plates, reinforced bolted slotted-in steel plates, and glued-in steel rods (GSRs) are the most common types of beam-column connections [6]. Recently, specially designed hybrid connections with a bi-linear moment-rotation curve have been developed, such as steel dampers with GSRs [7] and ductile steel link with self-tapping screw connections [8]. With the increase in the availability of ductile and high moment-capacity joints, timber-based moment-resisting frames (MRFs) can be utilized effectively, either independently or in combination with other LLRSs. Reinforced concrete (RC) dual systems that comprise shear-wall and MRFs are common in practice [9,10]. Usually, in these systems, the shear-walls are designed to resist the entire lateral loading, while the MRFs are barely designed to handle the gravity loads [9]. However, studies (e.g., [9-12]) have shown that in addition to jointly supporting the gravity loading, the MRF increases the lateral stiffness of the structure and contributes to resisting the applied loads.

The benefits of increasing stiffness depend on the relative stiffnesses of the shear-walls and frames and the height of the building [9]. Under the action of a lateral load, the natural tendencies of the wall and frame are to deform with different configurations [9,10], with the shear-wall in combined rocking-flexure and the MRF in shear (Figure 1(a)). With this different mode of deformation, one system tries to pull the other, resulting in a horizontal interaction between them. Consequently, an interaction develops, and negative-moment is created on the upper part of the MRF system (Figure 1(b)) [9]. This observation is significantly true for tall timber buildings where the shear-walls are relatively flexible. To control this detrimental effect, several studies have been made on RC wall-frame systems (e.g., [10-13]). Two of the most common strategies are wall curtailments and storey stiffening. By curtailing or terminating the shear-walls, above a certain storey level, the negative moments and shear forces in the wall can be reduced, while the top deflection is negligibly affected [10-12]. Alternatively, the shear rigidity of the frame system can be increased by either infilling or bracing one or more bays of the frame system [10,13].



(a) *(b)*

Figure 1. CLTW-GMRF system: (a) deformation mode, (b) moment distribution.

Similar to the RC wall-frame system, the timber frames in timber shear-wall frame systems have been designed for gravity loads only. Recently, prepared for British Columbia (BC) Forestry Innovation Investment Ltd., Tesfamariam and Teweldebrhan [5] explored the feasibility of the CLT balloon shear-wall and Glulam moment resisting frame (CLTW-GMRF) dual system as a LLRS in timber buildings and studied the potential interaction between the two systems. In their study, the CLT balloon shear-walls and the GMRF systems were designed to resist a predetermined seismic bending moment. Figure 1(b) illustrates the distribution of moment throughout the height of a 10-storey CLTW-GMRF system. As can be seen from the figure, the targeted moment proportion (MP), which is 50% to 50% (wall to frame) in this particular case, is achieved by properly designing the system. Furthermore, the figure shows that the CLT shear-wall is carrying a negative moment at the top 5 storeys, while the frame is subjected to an additional bending moment (larger than the seismic moment). Thus, this paper extends the study on CLTW-GMRF systems and investigates the effect of wall curtailments and infills configurations in the general behavior of a typical 10-storey CLTW-GMRF system. Linear static, nonlinear static, and dynamic analyses are used to examine a total of 24 systems with different wall configurations and address the aforementioned objective. First, linear static analysis is performed using ETABS software, and the changes in moment distribution of each system and the beam-column joint moment demands, are computed and compared. Second, a two-dimensional (2D) numerical model of the system is developed in OpenSees, and the performance of different systems is examined using nonlinear static analysis. Finally, 30 ground motion (GM) records (bi-directional) are selected to represent the seismicity of Vancouver, Canada, and are used to study the performance of the systems under seismic excitations. The impacts on interstorey drift ratio, residual drift ratio, and peak floor acceleration are examined, collapse fragilities are developed, collapse margin ratios are computed, and detailed comparisons are provided.

BUILDING DETAILS AND DESIGN CONSIDERATION

System information

The proposed CLTW-GMRF system comprises glulam beams and columns, CLT floor and shear-wall panels, moment-resisting beam-column joints, and buckling-restrained braces (BRB) hold-downs. While the 3D diagram of the system can be found in [5], Figure 1(a) illustrates the seismic resisting structural system of the building; the peripheral glulam frame elements and CLT shear-wall. The total height of the building is 30 m, and the length of each bay and CLT balloon shear-wall is 4 m. The designed dimension of beams, columns, and CLT shear-wall (thickness) are 265 by 418 mm, 365 by 494 mm, and 245 mm, respectively.

Case study building types

Categorized into 6 groups, a total of 24 CLTW-GMRF systems with different wall curtailments and infill configurations are examined in this study (Table 1, Figure 2). Group 1 buildings are introduced to study the effect of wall curtailments (at different storey levels) on the moment demand, base shear strength, and seismic performance of the system. Buildings in groups 2 and 3 are developed to examine the effect of one storey wall-infill, when the infill is distributed horizontally and vertically, respectively. Group 4 buildings are considered to study the effect of wall-infills in two storeys followed by groups 5 and 6, that are developed to examine the combined effect of wall curtailment and infill configurations. It should be noted that while the location of wall curtailments and infill configurations in groups 1 and 2 are decided based on previous studies [10-13], the rest groups are developed based on the outcome of their preceding cases. Moreover, it is worth to mention that CLT panel of length 2 m and 105 mm is used as infill wall.

Table 1. Case study description.

Group	No. of models	Walls curtailed storey levels	Infilled bay storey levels	Remark
Baseline	1	-	-	-
1	5	6^{th} – 10^{th}	-	Wall curtailed at storey and above
2	3	-	5^{th}	Outer, inner, and both bays are infilled
3	5	-	4^{th} – 8^{th}	1 storey infilled at outer bays
4	3	-	4^{th} & 6^{th}, 5^{th} & 7^{th}, and 6^{th} & 8^{th}	2 storeys infilled at outer bays
5	5	10^{th}	4^{th} – 8^{th}	1 wall curtailed and 1 storey infilled
6	3	10^{th}	4^{th} & 6^{th}, 5^{th} & 7^{th}, and 6^{th} & 8^{th}	1 wall curtailed and 2 storeys infilled

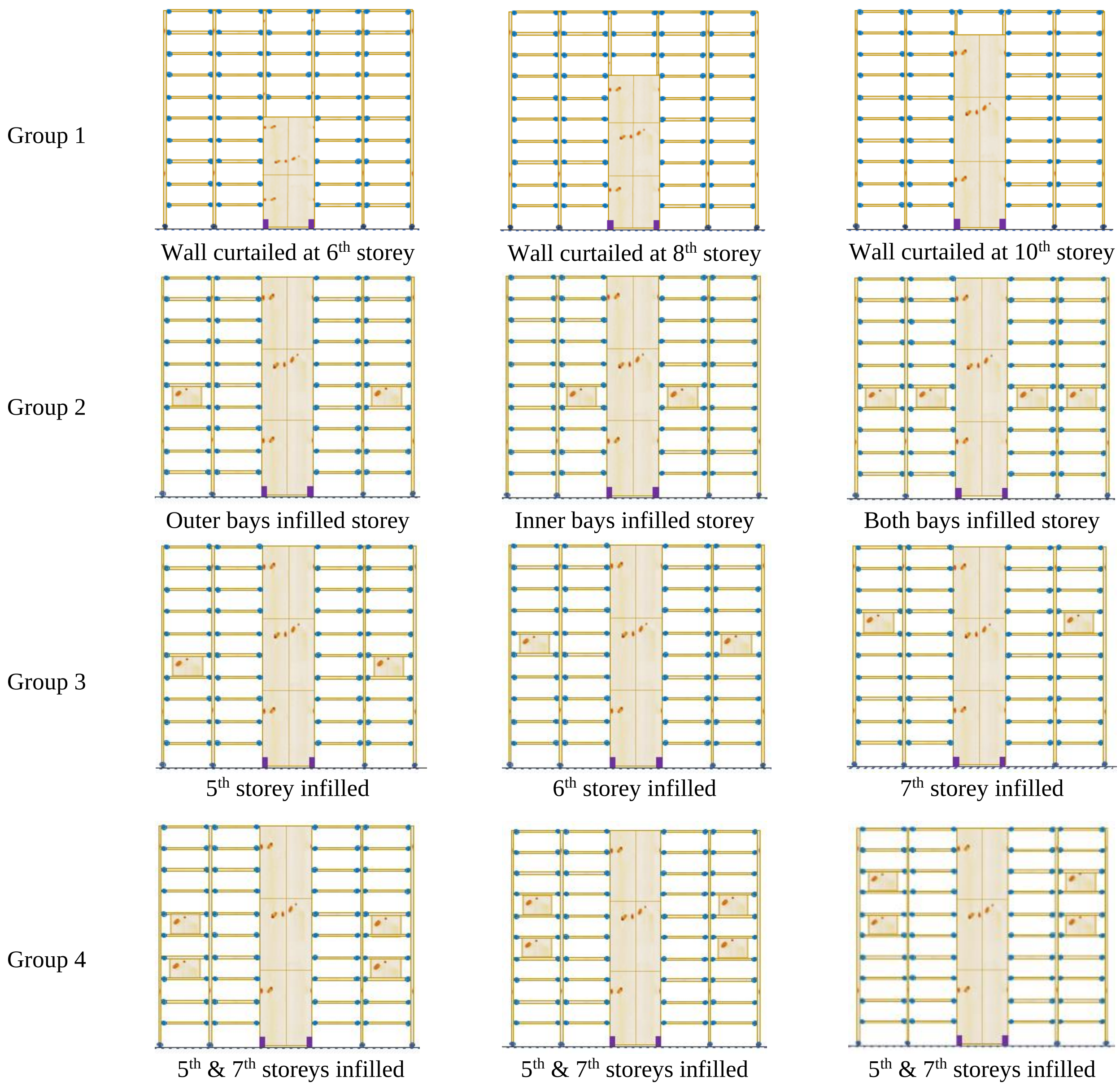


Figure 2. Case study buildings (Note: Some of the case study buildings are not presented in this figure).

Seismic design

Linear and nonlinear analysis models can be used to analyze the wall-frame systems [9]. In this study, targeted MP between the two systems (CLTW and GMRF) is chosen, and the seismic demands of the dual system is computed using equivalent static force procedure (ESFP) [14]. Based on the targeted MP value, continuous medium method (CMM) is used to determine the seismic demand in each component of the system. ETABS program is then used to analyze the system. The force parameters of the glulam frame elements and CLT wall piers are then extracted and designed following the CSA 086-19 [15] standard. The complete seismic design procedure, along with a numerical example, is provided in Tesfamariam and Teweldebrhan [5].

NUMERICAL MODEL AND GROUND MOTION SELECTION

A finite element model of the system, shown in Figure 3, is developed in OpenSees. The CLT panels are modelled using *ElasticIsotropic* material and *quad* elements [16]. The glulam frame elements are modeled using the *elasticBeamColumn* element feature [17]. The beam-column and column-base joints are modelled as *zeroLength* nonlinear rotation springs using OpenSees *Steel02* and *Pinching4 uniaxialmaterials*, respectively. Experimental data from existing study [7], for the beam-column joint response, is used and calibrated in OpenSees (Figure 3(b)). To satisfy the high axial demand, BRB hold-downs are utilized, and are modeled using *zeroLength Steel01 uniaxialMaterial* [18-20]. The contact between the CLT wall and the base is modeled using *uniaxial elastic notension (ENT)* material and a large elastic stiffness value is assigned to the *ENT* spring under compression. Steel slit dampers, modelled using *RambergOsgoodSteel*, are used as connectors between the infill CLT wall and beam elements. Besides, to take account the effect of P-delta, a leaning column is introduced and modeled as *elasticBeamColumn* element. Elastic *UniaxialMaterial* and *truss* elements are used to link the CLTW-GMRF and leaning column and transfer the P-delta effect. The detail modeling parameters can be obtained in Tesfamariam and Teweldebrhan [5].

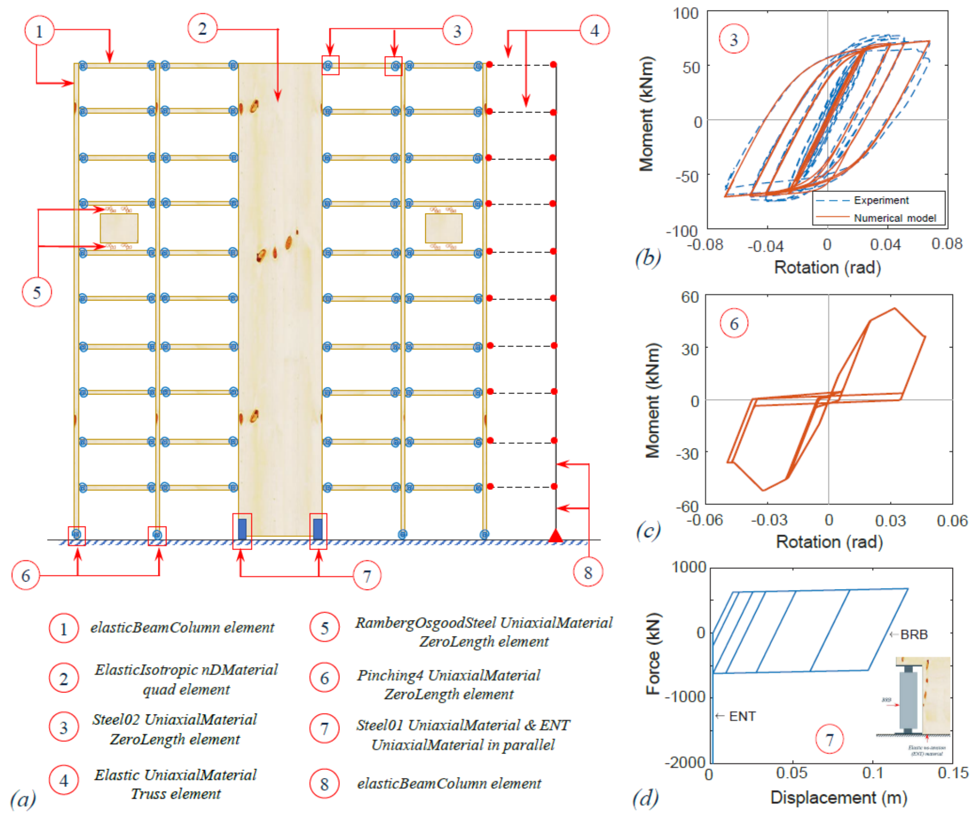


Figure 3. Numerical model:(a) CLTW-GMRF system, (b) beam-column joint, (c) column-base joint, and (d) hold-down.

Based on the developed OpenSees model, eigen value analysis is performed, and the first three fundamental periods are determined (For the baseline case: 1.93, 0.42, and 0.18 s). An in-house probabilistic seismic hazard analysis tool, developed by Tesfamariam et al. [21] based on the recently released 6th generation NBC 2020 seismic hazard model, is used to select 30 pairs of GM records at the anchor period of 2.0 s corresponding to a 2475-year return period. The selected GM records possess the key features of seismic sources of Vancouver, BC – Canada. Vibration periods of 0.25 s and 4.0 s are considered as the lower and upper limits for the GM selection, and the match is satisfied within these lower and upper limits vibration period ranges.

RESULT AND DISCUSSION

As discussed earlier, elastic analyses methods (CMM and ESFP) are used to determine the preliminary geometries and seismic demands of the system. For a more accurate evaluation of the system's performance, nonlinear analyses methods are used alongside the design based on elastic analyses. Results for the aforementioned analyses are categorized as linear static, nonlinear static, and dynamic analyses and are presented in the subsequent subsections.

Linear static analysis

The CMM, based on a closed form solution, is a simplified elastic method that can be used to analyze and design the preliminary geometries of the system [9]. Once the total base shear is determined based on ESFP and the force is distributed throughout the height of the dual system, the closed form solutions are used to determine the seismic share (moment and shear) for each of the system's LLRS components (the shear-wall and frame). ETABS program is then used to analyze the system and compute the change in force demands. Specifically, in this analysis step, the change in the bending moment distribution throughout the height of the two LLRSs and the moment demand in the beam-column joints is computed.

Effect of shear-wall curtailment

As presented in Table 1, 5 CLTW-GMRF systems are developed by terminating the CLT shear-walls at different storey levels (6th to 10th). Figures 4(a) and 4(b) illustrate the distribution of moment demand in the shear-wall and frame system, respectively. Three observations can be noted from the figure. First (Figure 4(a)), the negative moment developed in the top 5 storeys of the baseline are now reduced into 1 to 3 storey levels, depending on the level of curtailment. For example, systems whose shear-wall curtailed at 6th and 10th storey have now 1 and 3 storeys with negative moment, respectively. Second (Figure 4(a)), the systems whose shear-wall curtailed at the 9th and 10th storey levels exhibited relatively a great reduction in the magnitude of the negative moment. Third (Figure 4(b)), systems whose shear-wall curtailed at the 6th and 10th storey levels developed higher bending moment in the bottom 6 to 7 storey levels of the GMRF system. From the 5 considered systems, the ones whose shear-walls curtailed at the 9th and 10th storey levels have shown a reduction in the moment demand of the GMRF system.

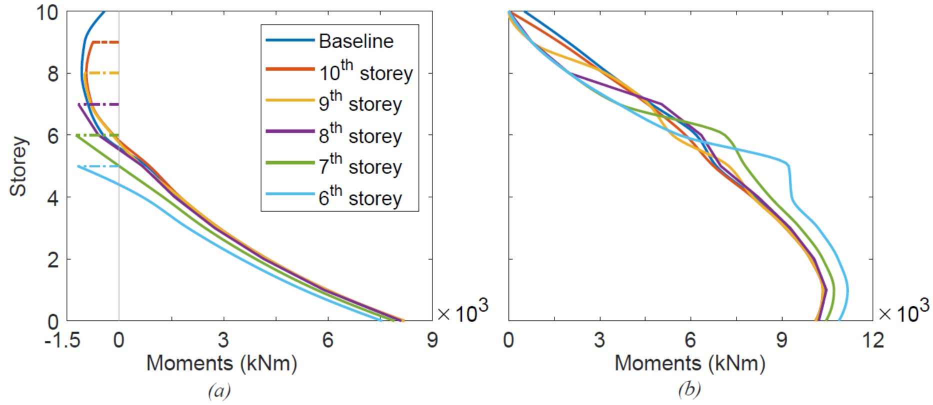


Figure 4. Bending moment demand distributions (group 1): (a) CLT shear-wall, (b) GMRF system.

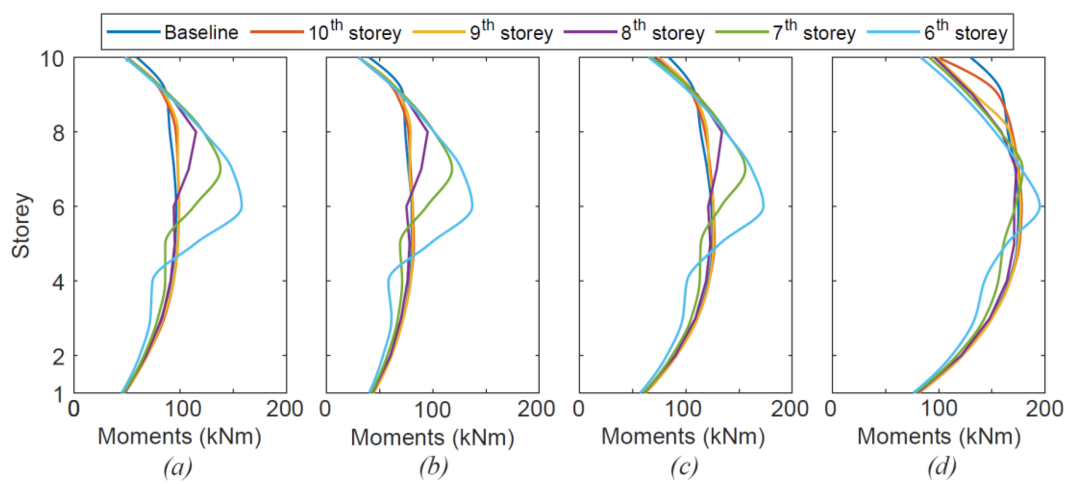


Figure 5. Moment demands at: (a) outer, (b) 1st interior, (c) 2nd interior, and (d) 3rd interior beam-column joints (group 1).

Figure 5 is prepared to have an idea on the change in the moment demand of the beam-column joint at different positions. Figures 5(a), 4(b), 4(c), and (d) represent the demand in the outer, first interior, second interior, and third interior beam-column joints, respectively (left to right for the left bays of the GMRF and right to left for the right bays of the GMRF in Figure 1(a)). Two important notes can be made from the figure. First, systems whose shear-wall curtailed at the 9th and 10th storey levels exhibited lower moment demand in almost all the location of the beam-column joints. The opposite is happening for the systems whose shear-walls are curtailed at the 6th to 8th storey levels. Second, the maximum moment demand in the beam-column is occurring at the shear-wall curtailment level of the system. It should be also noted that comparing to those located in the outer bays, the beam-column joints located at inner bays have higher demands (in all the cases including the baseline).

Effect of one storey wall-infills

Two groups of systems are analyzed to examine the effect of one storey wall-infills. In the first group (group 2 in Table 1), the 5th storey bays are infilled in three ways; first at the 2 outer bays, second at the 2 inner bays, and at last in all the 4 bays. From Figure 6(a), it can be noted that all the systems have eliminated the negative moment at the top storey levels. However, systems whose inner and both bays infilled create negative moments in the 3rd and 4th storey levels. Accordingly, higher moment demand is observed in the GMRF system (Figure 6(b)). Therefore, to eliminate the negative moment in the wall system, it is advisable to infill the outer bays. Based on this observation, the second group (group 3 in Table 1) are analyzed by infilling the outer bays at 4th to 8th storey of the CLTW-GMRF system. As can be noted from Figure 7(a), systems whose outer bays infilled at the 4th to 6th storey levels have reduced both the location and magnitude of the negative moment observed in the baseline system. However, from Figure 7(b), it can be seen that all the infill configuration led to an increase in the moment demand of the frame system. This observation is relatively higher for the systems whose bays infilled at the 4th and 5th storey levels.

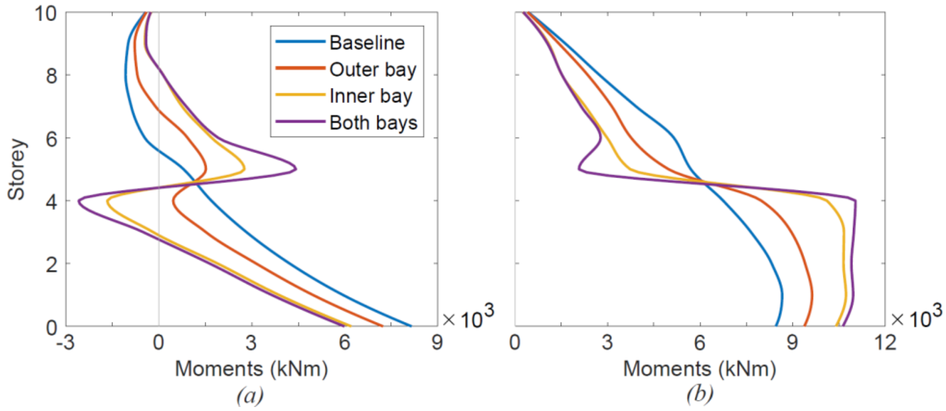


Figure 6. Bending moment demand distributions (group 2): (a) CLT shear-wall, (b) GMRF system.

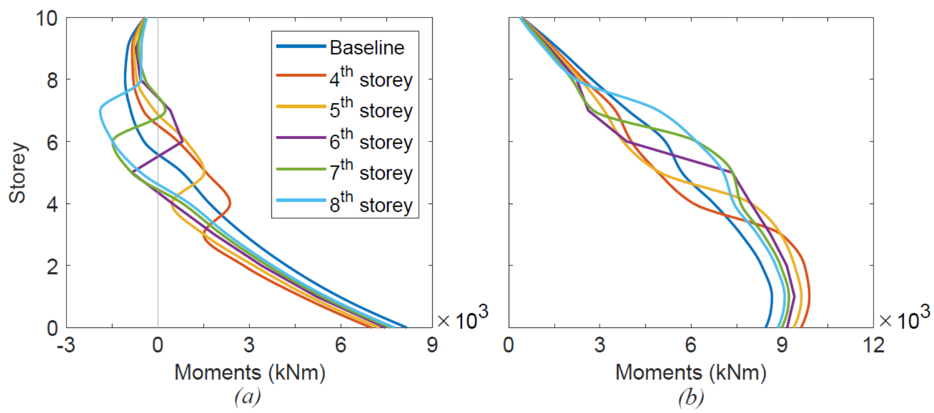


Figure 7. Bending moment demand distributions (group 3): (a) CLT shear-wall, (b) GMRF system.

Figure 8 shows the distribution of moment demand at the four beam-column joint locations throughout the height of the buildings. As can be seen from the figure, the beam-column moment demands at the outer and first interior joints have reduced by 25% in almost all the systems examined (Figures 8(a) and 8(b)). Even though it is not as significant as those at the support and first interior joints, a reduction in moment is also observed in the second interior joint location (Figures 8(c)). The reduction at the third interior joint can be considered as an intermediate with a reduction of 40 kNm at the middle height of the buildings. From all the systems analyzed, the one infilled at the 5th storey level has shown better improved (both in Figures 7 and 8).

Effect of two storey wall-infills

Additional three systems are analyzed to examine the effect of wall-infills at two different storey levels (4th & 6th, 5th & 7th and 6th & 8th levels). Those locations are selected based on the results observed in the preceding subsection. Note that in all of these 3 systems, the outer bays are selected to be infilled. From Figure 9(a), it can be noted that systems whose bays infilled at the

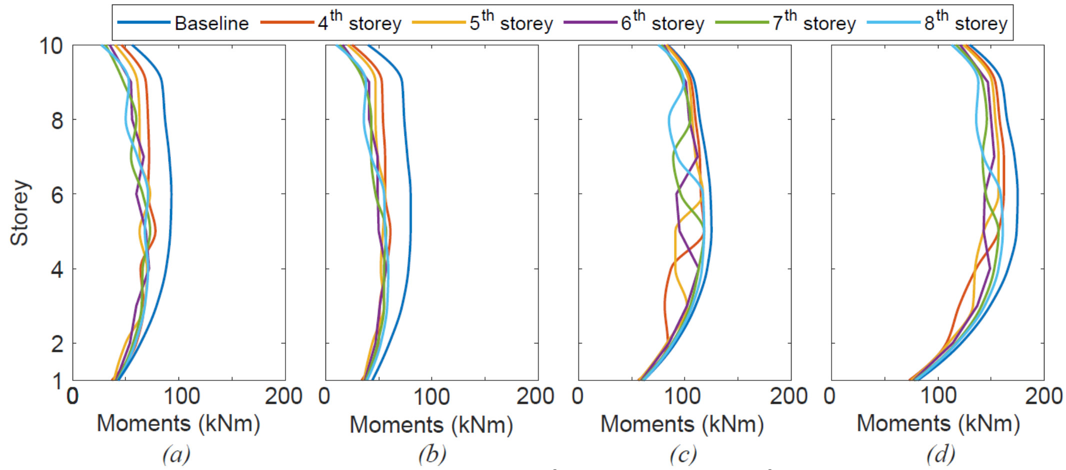


Figure 8. Moment demands at: (a) outer, (b) 1st interior, (c) 2nd interior, and (d) 3rd interior beam-column joints (group 3).

4th & 6th and 5th & 7th have reduced both the location and magnitude of the negative moment observed in the baseline system. However, the system whose outer bays infilled at the 6th & 8th storeys has shown to create negative moments in the upper 6 storeys levels. Moreover, larger negative moment is observed in the described system.

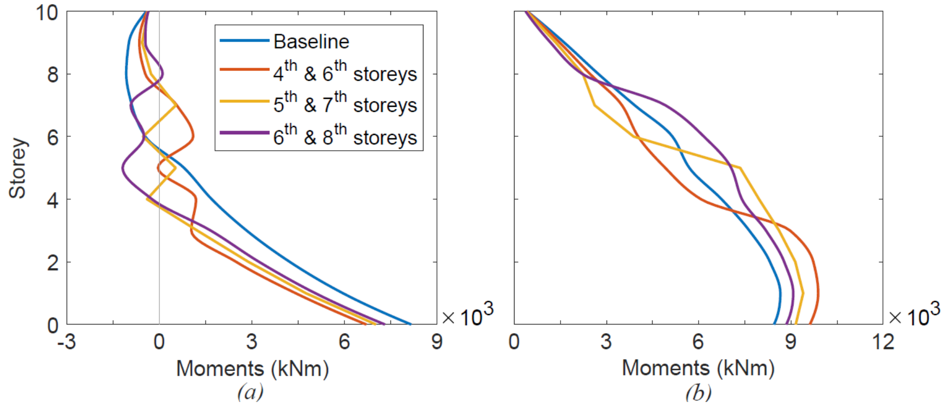


Figure 9. Bending moment demand distributions (group 4): (a) CLT shear-wall, (b) GMRF system.

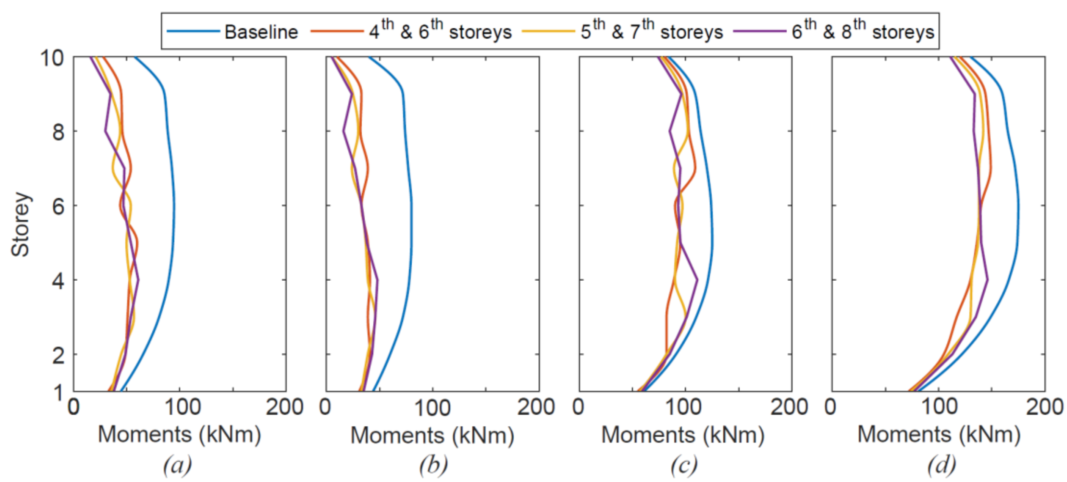


Figure 10. Moment demands at: (a) outer, (b) 1st interior, (c) 2nd interior, and (d) 3rd interior beam-column joints (group 4).

Figure 10 shows the distribution of moment demand at the four beam-column joint locations. As can be seen from the figure, the beam-column moment demands at the outer and first interior joints have reduced by 40 to 50% at the location of the maximum moment demand (Figures 10(a) and 10(b)). The corresponding reduction of moment demands at the second and third interior joint ranges 20 to 25% (Figures 10(c) and 10(d)).

Effect of combined wall curtailment and infill configurations

As described in Table 1, two groups (group 5 and 6) are analyzed to examine the effect of the combined wall curtailment and infill configurations. The systems with one and two storey wall-infill configurations (explained in the preceding 2 subsections) are reassessed by curtailing their 10th storey shear-wall. A small difference is observed in the linear static analysis results comparing to the systems with full height walls. Thus, the result presented in the preceding 2 subsections can be generalized and used here. Their nonlinear analysis results, however, is presented in the subsequent sections of this paper.

Nonlinear static analysis

The study on the effect of the wall curtailments and infill configurations is further examined in this section using monolithic pushover analysis. Specifically, the effect of the different configurations on the base shear strength, collapse drift ratio, initial, and post-yielding stiffnesses are discussed. Figure 11 presents the findings of the analyses. The systems are pushed up to collapse, which is considered to occur either at global level due to system instability or the local failure (in this system, failure of the weakest link or the beam-column joints, detail discussion is provided in Tesfamariam and Teweldebrhan [5]). Figure 11(a) illustrates the effect of wall curtailments (only 8th – 10th storey). As can be seen from the figure, systems whose wall-curtailed at the 8th and 9th storey levels collapse at 3.75 and 3.91% drift ratios, which is relatively less comparing the collapse drift ratio (≈4.81%) for the systems whose wall-curtailed at 10th storey and the baseline case. Moreover, the system whose wall-curtailed at the 8th storey exhibits lower initial and post-yielding stiffnesses with lower ultimate base shear strength.

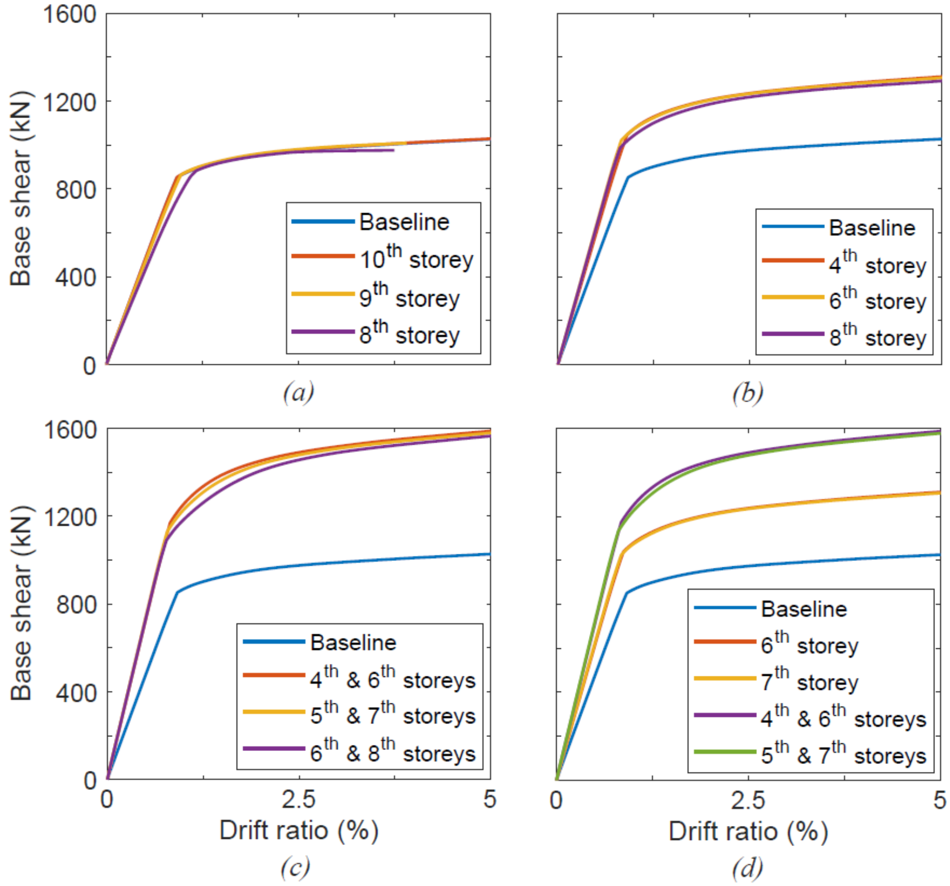


Figure 11. Monolithic pushover analysis results for systems with: (a) wall curtailments, (b) 1 storey wall-infills, (c) 2 storey wall-infills, and (d) combined wall curtailment and infill configurations.

On the other hand, the system whose wall-curtailed at the 10th storey exhibited almost the same ultimate base shear strength and drift capacity along with the same initial and post-yielding stiffnesses. From the rest of the figures, Figures 11(b) to 11(d), three observations can be noted. First, the ultimate base shear strength, ultimate drift capacity, initial and post-yielding stiffnesses increases with the increase in the number of infilled storeys (i.e. two storey wall-infilled system out-performs the one storey infilled systems, see Figure 11(d)). Second, systems stiffened at almost middle height out-performs those that are stiffened at lower and higher storey levels (e.g., 5th and 6th storey, and 4th & 6th and 5th & 7th infilled systems from one- and two-storey infilled systems showed higher performance). Finally, in all the case (Figures 11(a) to 11(d)), the same performance is observed between the systems that have full height shear-walls and the systems with their 10th storey shear-wall curtailed.

Nonlinear dynamic analysis

Nonlinear dynamic analyses, both at maximum considered earthquake (MCE) level and incremental dynamic analyses (IDA), are used to assess the nonlinear response of the systems under the action of seismic excitation. The analyses are carried out using a total of 60 GM records and the results are presented in the subsequent subsections.

Peak responses under MCE level

Three engineering demand parameters, Maximum inter-storey drift ratio (MaxISDR), residual inter-storey drift ratio (ResISDR), and peak floor acceleration (PFA), are selected to explore the effect of wall curtailments and infill configurations on the seismic performance of the system. Figures 12 to 14 demonstrate the effects of wall curtailments, one storey and two storey wall-infill configurations, respectively.

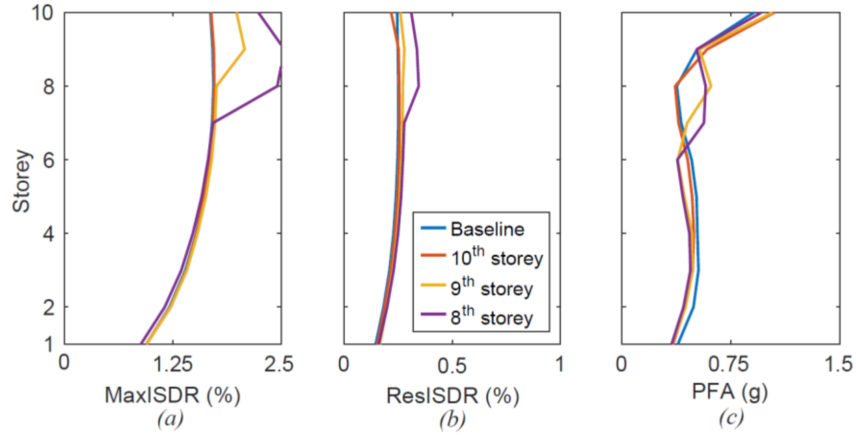


Figure 12. Nonlinear dynamic analysis results (group 1): (a) MaxISDR, (b) ResISDR, and (c) PFA.

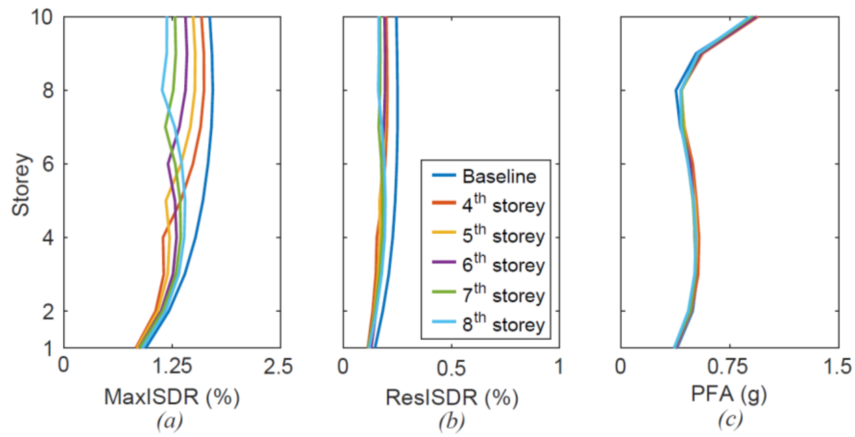


Figure 13. Nonlinear dynamic analysis results (group 3): (a) MaxISDR, (b) ResISDR, and (c) PFA.

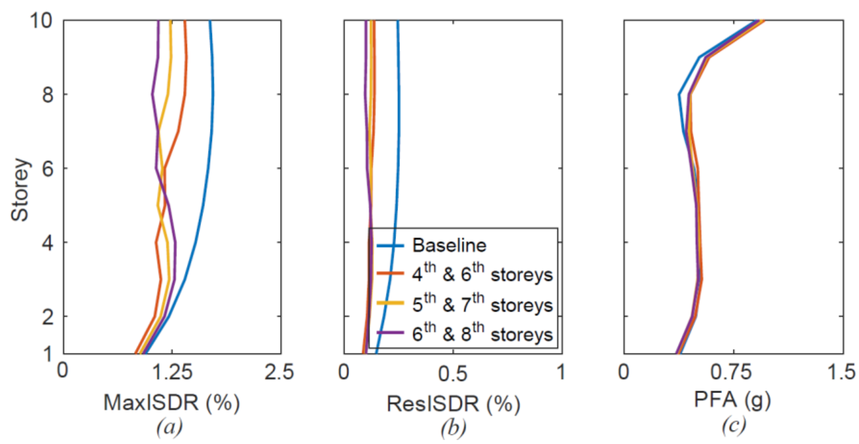


Figure 14. Nonlinear dynamic analysis results (group 4): (a) MaxISDR, (b) ResISDR, and (c) PFA.

Comparing to the baseline system, systems whose shear-wall curtailed at 8th and 9th storey exhibit higher MaxISDR, ResISDR and PFA values (Figure 12). The system whose shear-wall terminated at the 10th storey exhibits the same MaxISDR, and relatively smaller and higher ResISDR and PFA values than the baseline, respectively. On the other hand, systems with one storey infilled walls have shown significantly lower MaxISDR and ResISDR values compared to the baseline (Figures 13(a) and 13(b)). From the 5 systems examined in this group, the systems whose outer bays infilled at the 5th to 7th storey have shown relatively smaller MaxISDR. The systems whose outer bays infilled at the 4th and 8th storey levels have relatively larger MaxISDR at the top and lower storeys, respectively (comparing to the other wall-infill configurations). However, a small increase in PFA is observed in all the one-storey wall-infilled systems comparing to the baseline. Similarly, systems with two storey wall-infills have shown much lower MaxISDR and ResISDR values (Figures 14(a) and 14(b)) and the one whose 5th & 7th storeys infilled shows the best performance.

IDA and collapse fragility curves

IDA is performed to examine the performance of the different CLTW-GMRF systems under the action of different seismic intensity levels. Based on a preliminary assessment, the GMs were scaled up until the intensity measure (spectral acceleration) value triggered the collapse of the building. The IDA results are then used to develop the collapse fragility curves and compute collapse margin ratio (CMR) of the different CLTW-GMRF systems. Fragilities curves are developed by computing the probability of collapse, $P(C)$, of the structure. Lognormal cumulative distribution function (CDF) is fitted and the method of moments is applied to obtain the parameters of the distribution function at collapse. These parameters (the mean and standard deviation) are then used as an input to define the CDF. The typical collapse fragility curves for the different CLTW-GMRF systems are shown in Figure 15. Four observations can be noted from the figure. First, as discussed earlier, the fragility functions reaffirm that systems with walls curtailed at the 8th and 9th storey level have higher P(C) or lower seismic performance. Although the system whose wall curtailed at the 10th storey level seemed to have same capacity at lower GM intensity level, it has got a higher P(C) at moderate to high GM levels compared to the baseline. Second, the systems that are stiffened with infills exhibit lower deviation (in all the infill configurations including when combined with curtailment). Third, the seismic performance of both one and two storey infilled systems is significantly improved (especially at the low to moderate GM intensity levels). Four, systems with combined effect have shown lower performance compared to those that are not curtailed.

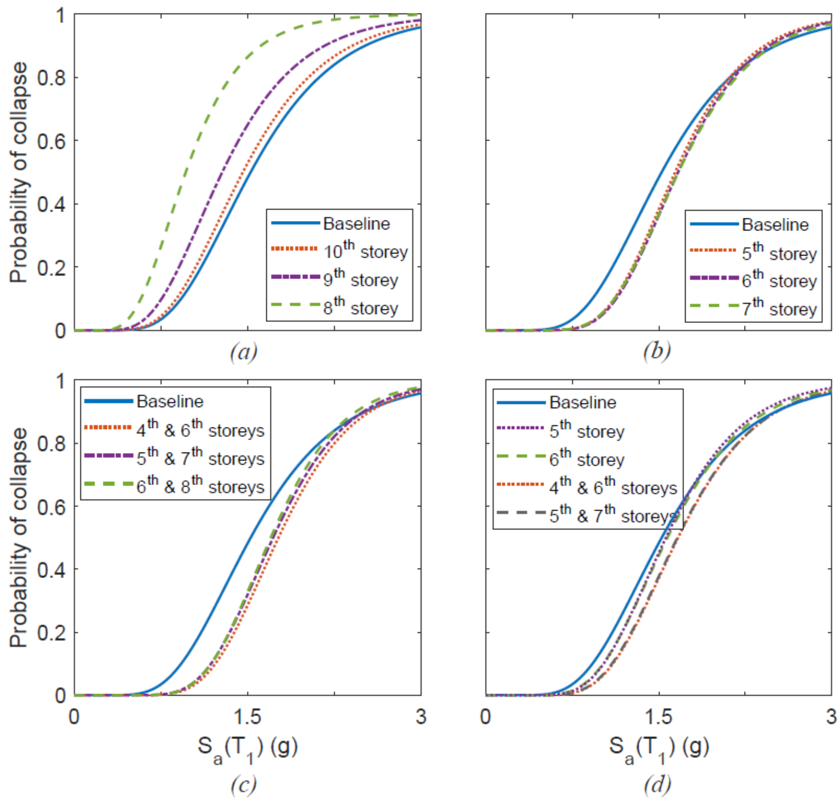


Figure 15. Collapse fragility curves for systems with: (a) wall curtailments, (b) 1 storey wall-infills, (c) 2 storey wall-infills, and (d) combined wall curtailment and infill configurations.

The aforementioned observations do not consider the change in the fundamental periods of the systems. However, the change in the periods of the system can greatly affected performance measures such as CMR. Up on considering the change in fundamental periods of the systems, the fragility parameters (including the CMR) are computed and are summarized in Table 2. From Table 2, four key observations are noted. First, it is demonstrated that the fundamental periods of the systems with curtailed walls is reduced and thus, their initial stiffness is increased. Second, the standard deviation or dispersion is lower for stiffened systems comparing to the baseline and the systems with curtailed walls. Third, by stiffening the system, the P(C) at MCE level can be reduced significantly (in the range of 53 to 99%). However, curtailment of shear-walls has increased the P(C) at MCE level. Fourth, there is small difference in CMR between the baseline and the stiffened systems (in the range of 4 to -8%). Systems with curtailed walls and two storey wall infills have lower CMR values.

Table 2. Effect of wall curtailments and infill configurations on fragility parameters.

Group	**Curtailed levels**	**Infilled levels**	**Fundamental periods**			**Fragility parameters**							
			T_1	**T_2**	**T_3**	$\bar{\mu}$	$\bar{\beta}$	S_{MT}	**P(C/MCE) %**	**ΔP(C/MCE) %**	$\hat{S}_{CT}$	**CMR**	**ΔCMR %**
Baseline	1	-	1.93	0.42	0.18	1.02	0.39	0.28	0.046	-	0.99	3.54	-
	10^{th}	-	1.91	0.43	0.25	0.97	0.39	0.28	0.071	-55	0.95	3.37	-4.74
1	9^{th}	-	1.90	0.54	0.29	0.85	0.41	0.28	0.348	-660	0.68	2.39	-32.4
	8^{th}	-	1.91	0.67	0.29	0.64	0.40	0.28	1.952	-4162	0.67	2.37	-33.1
	-	4^{th}	1.71	0.41	0.18	1.07	0.31	0.31	0.002	95	1.07	3.48	-1.62
	-	5^{th}	1.67	0.42	0.17	1.10	0.30	0.31	0.002	97	1.11	3.55	0.44
3	-	6^{th}	1.66	0.41	0.17	1.11	0.30	0.31	0.001	97	1.12	3.58	1.13
	-	7^{th}	1.67	0.40	0.18	1.11	0.31	0.31	0.003	95	1.15	3.68	3.98
	-	8^{th}	1.69	0.39	0.18	1.10	0.32	0.31	0.004	92	1.12	3.63	2.48
	-	4^{th} & 6^{th}	1.54	0.41	0.17	1.18	0.29	0.33	0.001	99	1.16	3.52	-0.44
4	-	5^{th} & 7^{th}	1.53	0.40	0.17	1.15	0.29	0.33	0.001	98	1.12	3.39	-4.15
	-	6^{th} & 8^{th}	1.55	0.39	0.17	1.13	0.28	0.33	0.001	99	1.13	3.46	-2.14
	10^{th}	5^{th}	1.66	0.43	0.25	1.04	0.33	0.31	0.014	69	1.05	3.36	-4.96
5	10^{th}	6^{th}	1.65	0.43	0.24	1.05	0.34	0.31	0.021	55	1.03	3.26	-7.85
	10^{th}	7^{th}	1.66	0.42	0.24	1.07	0.35	0.31	0.022	53	1.07	3.40	-3.83
	10^{th}	4^{th} & 6^{th}	1.53	0.42	0.24	1.11	0.32	0.33	0.009	81	1.12	3.39	-4.31
6	10^{th}	5^{th} & 7^{th}	1.52	0.42	0.24	1.11	0.33	0.33	0.011	76	1.08	3.26	-7.96
	10^{th}	6^{th} & 8^{th}	1.55	0.41	0.24	1.10	0.31	0.33	0.005	90	1.08	3.28	-7.15

CONCLUSIONS

In the modern construction industry, timber-based lateral load resisting systems (LLRSs) are becoming popular due to urban densification and demand for sustainable materials. With the increase in the availability of ductile and high moment-capacity joints, timber-based moment-resisting frames can be utilized effectively. CLT shear-wall and Glulam moment resisting frame (CLTW-GMRF) is a recently completed research project. The outcomes of the research demonstrated that due to the different mode of deformation under seismic forces, the two LLRSs interacts and negative-moment is created in the upper part of the GMRF system. To control this detrimental effect, in this study, two strategies (wall curtailment and storey stiffening) are utilized and a total of 24 10-storey CLTW-GMRF systems are developed. Linear static, nonlinear static, and dynamic analyses are conducted and the change in the bending moment distribution of each systems and the beam-column joint moment demands are computed. The impacts on interstorey drift ratio, residual drift ratio, and peak floor acceleration are examined, collapse fragilities are developed, and collapse margin ratios are computed. The following conclusions can be made from this study:

- Depending on the level of curtailment, the negative moment developed in the wall-frame system can be eliminated or reduced. Systems with curtailed walls (at the 6^{th} to 9^{th} storey) have developed higher bending moment demand in the frame system, and exhibit lower ultimate base shear strength and drift capacities. Besides, they have shown higher P(C) at MCE level and lower CMR values. On the other hand, the system whose wall curtailed at 10^{th} storey exhibit almost the same base shear strength, drift capacity, initial stiffness, MaxISDR, ResISDR and PFA as the baseline.
- Infilling the outer bays has shown an improvement in the overall mechanics of the system comparing to infilling the inner or both bays. By infilling the outer bays at one storey level, the negative moment of the wall can be eliminated. Almost all the examined systems have enhanced the behavior of the baseline system. With these systems, the beam-column moment demands at the outer and first interior joints can be reduced by 25%. Moreover, these systems can increase the ultimate base shear strength (by 25%), drift capacity, initial stiffnesses, and CMR of the system. Besides, these systems have shown significantly lower MaxISDR and ResISDR values compared to the baseline.
- Systems whose outer bays infilled at two storeys are most effective in reducing the negative moment of the baseline system. With these systems, the beam-column moment demands at the outer and first interior joints have reduced by

40 to 50% and the ultimate base shear strength can be increased by 50%. They also increase the ultimate drift capacity and the initial stiffness of the system. Moreover, they have shown much lower average MaxISDR and ResISDR values.
- Compared to the baseline, both the one and two storey infilled systems have lower P(C) with higher and lower CMR values, respectively.
- Almost no change is observed (in both the static analyses) when curtailing the 10th storey of the infilled wall systems. However, the systems have got higher P(C) at moderate to high GM levels compared to the baseline and infilled systems. Thus, they exhibit lower CMR value.

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