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SEISMIC DEMANDS IN MULTI-STOREY CROSS-LAMINATED TIMBER (CLT) STRUCTURES

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Abstract: *There is a growing global interest on multi-storey building construction with cross-laminated timber (CLT). This has been driven by the numerous benefits offered by CLT over other traditional construction materials like its minimal environmental impact, outstanding structural performance, and accelerated construction time resulting in significantly reduced costs. Besides, the dynamic response of CLT buildings has been the subject of numerous experimental as well as numerical studies during the last 20 years. However, most of these efforts have focused on the identification of suitable behaviour factors and strength limits and no comprehensive assessment has been conducted with regards to the expected levels of inelastic displacement and force demands in CLT buildings. Likewise, the crucial role played by ground-motion frequency-content has been routinely disregarded. This paper investigates the inelastic seismic demands in multi-storey CLT structures with emphasis on deformations, forces and accelerations. The influence of structural as well as ground-motion frequency content on the drift, acceleration and shear demands in tall CLT buildings is carefully examined and rigorous predictive models are proposed.*

Introduction

Cross-laminated timber (CLT) has numerous advantages over conventional building materials including minimal environmental impact, efficient structural performance, and reduced construction time and cost. Besides, multi-storey CLT construction is an appealing and environmentally responsible building option with a significant potential to optimize the use of the land, particularly in emerging urban areas where more than 15 million people are currently living (Goncalves and Umakoshi, 2010). Furthermore, the inherent in-plane stiffness of CLT and the possibility of designing ductile joinery offer an attractive alternative for areas of high seismicity.

Numerous experimental and numerical studies have been performed to study the seismic response of CLT buildings. One of the most comprehensive experimental projects on the seismic behaviour of CLT construction (SOFIE project) was carried out by CNR-IVALSA. As part of the SOFIE project, shear-wall tests on various connection and panel arrangements as well as pseudo-dynamic tests on full-scale 3- and 7-storey CLT buildings subjected to real strong ground-motion records on a shaking-table (Ceccotti *et al.*, 2006; Ceccotti *et al*., 2013) were conducted. Other experimental efforts aimed at studying the lateral response of CLT buildings and panel assemblies were performed by Málaga-Chuquitaype *et al.* (2016), Yasumura *et al*. (2015), Popovski and Gavric (2015), and Flatscher and Schickhofer (2015). These experimental studies have demonstrated that appropriately assembled and well-designed CLT structures have a relatively good seismic response.

A large number of numerical studies has also been conducted on the seismic response of CLT buildings. Most of these studies have been circumscribed to the calibration of numerical models against experimental results, and the characterisation of appropriate response modification factors for low- to mid-rise CLT buildings (Ceccotti and Sandhaas, 2010; Pozza and Scotta, 2015;

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Rinaldin and Fragiacomo, 2016). However, no comprehensive assessment has been conducted with regards to inelastic drift demands in multi-storey buildings. Furthermore, the influence of frequency content of the ground-motion that has been shown to be of paramount importance in seismic design and assessment of buildings (Málaga-Chuquitaype, 2015, Málaga-Chuquitaype *et al.*, 2019), has been usually ignored in past research studies with CLT. To this end, this paper provides a detailed account of recent studies on the seismic inelastic demands in multi-storey CLT buildings and the influence of frequency content of the ground-motion.

Structural Configuration and Design

Seismic design

This section offers a detailed account of the seismic design of prototypical multi-storey CLT buildings covering a wide range of structural characteristics. In the absence of specific design guidance on the seismic design of CLT structures, the general principles and considerations of capacity design and failure mode control outlined in Eurocode 8 (CEN, 2004) were followed in this study.

Figure 1 shows a plan view and elevation of a 6-storey CLT building. The same plan layout was utilized for all building heights (i.e. number of storeys n = 6, 8, 12, 16, 20) analyzed in this research. 5-layered CLT panels with varying thicknesses between 95 mm to 200 mm were employed for shear walls and 5-layered 200 mm thick were used for ceilings and the roof. The total design dead load was calculated considering all finishing and insulation components while a superimposed load of 2.00 kN/m^2 (residential buildings in Service Class A) was adopted for the roof and floor slabs. The corresponding building weight and seismic mass were determined as a combination of the total dead load and 30% of the superimposed load. The total superimposed dead load was determined considering all finishing and insulation components while a superimposed live load of 2.00 kN/m^2 (residential buildings in Service Class A) was adopted for the roof and floor slabs.

The Eurocode Type 1 response spectrum (for high seismicity areas) was adopted for the design of all buildings with soil type C conditions and a reference PGA of $\alpha_{gR} = 3.0$ m/s^2 with importance factor of $\gamma_i = 1.0$. C24 timber strength class (according to BS EN 14081-1:2005) was selected for all CLT panels. It is important to note that a range of behaviour factors were adopted in this study to evaluate the effect of different design assumptions on the inelastic response of CLT buildings. Additionally, a number of joint densities, namely m = {0, 1, 2, and 3} were employed to investigate the influence of wall fragmentation on the lateral response of CLT buildings. The number of connectors and vertical joints were determined to comply with capacity design principles. A detailed account of the type and distribution of structural timber connections can be found elsewhere (Demirci *et al.*, 2018; Demirci, 2019).

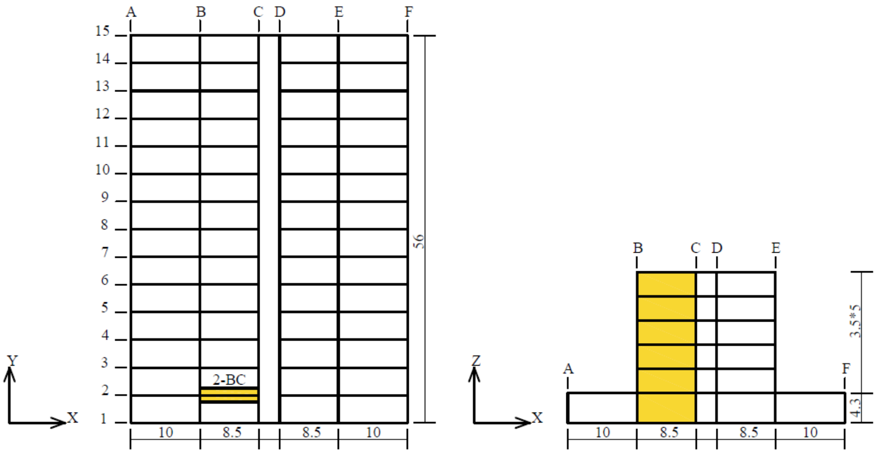


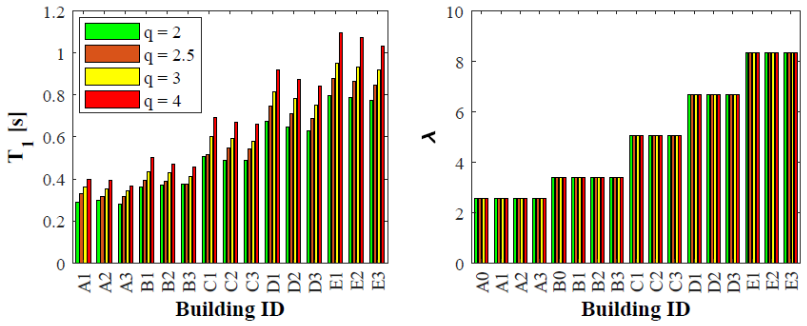
Figure 1. Plan view and elevation of the 6-storey CLT building. Dimensions in [m].

Structural characteristics

A large set of numerical models was built to perform a detailed parametric investigation on the seismic demands in multi-storey CLT buildings. To this end, the associated key structural characteristics for the building dataset developed can be summarized as follows:

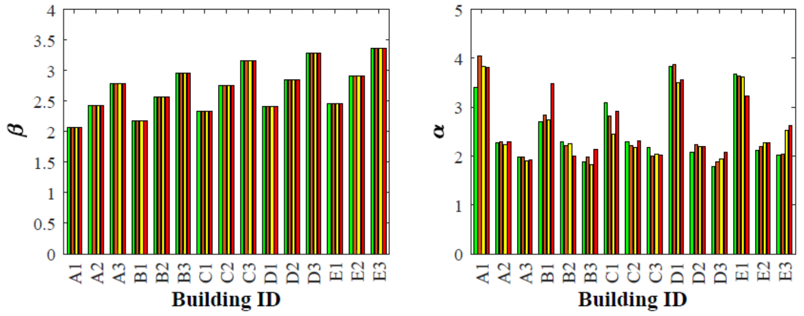
1. The fundamental period, T_1, is the structural period associated with the first natural mode of vibration. The fundamental periods for multi-storey CLT buildings considered range from 0.3 s to 1.0 s.
2. The building aspect ratio, λ, is the ratio between the height and the width of the building façade (H/B). The slenderness of the buildings analyses range from 2.56 to 8.33.
3. The joint density parameter, β, is the ratio between all fastening lines in the wall (P_0) and the perimeter of the wall (P) (Pozza and Trutalli, 2017). The distribution of joint density parameter ranges between 2.06 and 3.36.
4. Plasticity resistance ratio, α, is calculated as the ratio between the ultimate base shear, V_u, when a plastic mechanism is developed in the structure against the yield base shear, V_1. The distribution of plasticity resistance ratio ranges from 1.8 to 3.8.

The distribution of main structural characteristics outlined above can be seen in Figure 2.



(a) Fundamental period, T_1.

(b) Building aspect ratio, λ.



(c) Joint density parameter, β.

(d) Plasticity resistance ratio, α.

Figure 2. Distribution of structural characteristics of the building configurations under study.

Numerical modelling

Modelling assumptions

Numerical models of the CLT buildings under study were developed in the open-source Finite Element framework OpenSees (McKenna, 2011). Figure 1 shows the CLT core section 2-BC under consideration. The adopted numerical modelling approach is depicted in Figure 3. CLT panels were modelled using linear elastic 4-node Quad elements. Non-linear zero-length and two-node link elements were employed to simulate structural joints. Isotropic material properties were assumed for CLT panels. It is important to note that Quad elements in OpenSees rely on a 2-degree-of-freedom per-node idealization and they only allow for linear geometric transformation. Therefore, to account for P-delta second order effects a leaning column comprising beam-column elements with 3-degree-of-freedom per-node was modelled and attached to the core CLT wall model. Equal-degree-of-freedom multi-point constraints (EdofMP) were defined to simulate the rigid diaphragm connection between the two model domains and the corresponding vertical loads were applied to the leaning column at each floor level. Tri-linear hysteretic material properties available in OpenSees were used to model structural joints such as angle brackets, hold-downs, and vertical joints between adjacent wall panels. All joints were calibrated against experimental outcomes (Gavric *et al.*, 2015) and analytical estimations. Furthermore, the numerical modelling approach described above was extensively validated against available test results involving single joint, single and a coupled CLT wall panels, and a 7-storey CLT building shaking table tests. To this end, all comparisons between numerical outcomes and experimental results were found to be in good agreement. Detailed information about this comparative assessment can be found elsewhere (Demirci *et al.,* 2018).

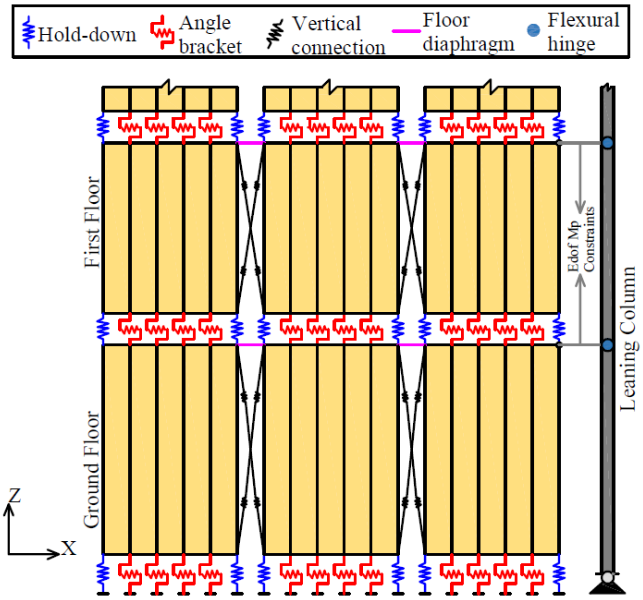


Figure 3. Schematic of numerical modelling approach for CLT buildings.

Ground-motion records and frequency content

A total of 1656 real strong ground-motions from 51 important seismic events with magnitudes, M_w, between 5.61 and 7.9 and with an average PGA of 1g were utilized for the non-linear dynamic analysis of buildings (Hancock *et al.*, 2008). Further information regarding the selected ground-motion records can be found in Demirci *et al*. (2018) and Hancock *et al*. (2008).

The mean period, a scalar parameter for the characterization of the frequency content of ground-motion (Rathje *et al.*, 1998; 2004), was adopted throughout this study. The mean period of the ground-motion was estimated as follows:

$$T_m = \frac{\sum_i C_i^2 * \frac{1}{f_i}}{\sum_i C_i^2} \text{ for } 0.25 \text{ Hz} \le f_i \le 20 \text{ Hz, with } \Delta f \le 0.05 \text{ Hz} \quad (1)$$

where C_i is the coefficient of the Fourier amplitude and f_i is the i^th^ frequency.

Seismic demand assessment

Nearly 100 000 non-linear response history analyses were performed by means of an array function that allows all analyses in parallel on the research computing facility at Imperial College London (HPC, 2017). The results of numerical simulations were then post-processed to obtain maximum displacements at the roof level, maximum drifts at each storey, maximum base shears, maximum storey shears, and maximum floor accelerations.

Seismic drift demands

This section outlines the seismic drift demands in multi-storey CLT buildings. To identify the seismic drift demands, two widely used parameters are employed:

- Global drift modification factor, δ_{mod}, is the ratio between the maximum roof displacement $\Delta_{\max}$ (obtained from nonlinear response history analysis) and the product of the roof yield displacement $\Delta_{1,\mathrm{roof}}$ (at first component yield obtained from nonlinear static pushover analysis with monotonically increasing lateral loads) times the corresponding design response modification factor, q. δ_{mod} is expressed as:

$$\delta_{\mathrm{mod}} = \frac{\Delta_{\max}}{q \cdot \Delta_{1,\mathrm{roof}}} \tag{2}$$

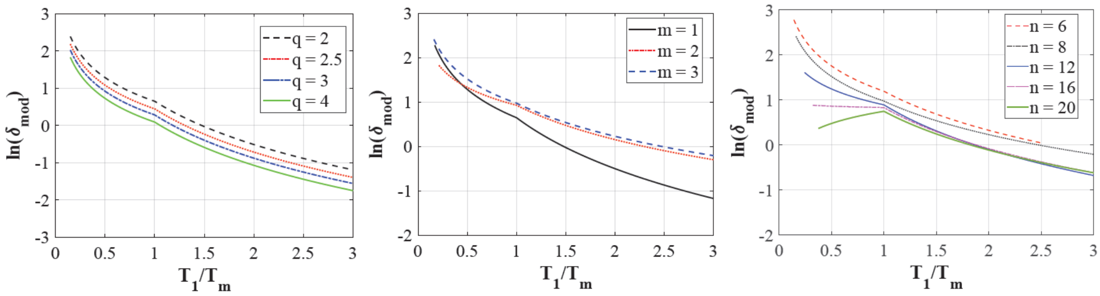
- Maximum drift modification factor, θ_{mod}, is the ratio between the maximum roof drift $\theta_{\max}$ (obtained from nonlinear response history analysis) and the product of the maximum inter-storey drift at first yield, $\theta_{1,\max}$ (obtained from nonlinear static analysis) times corresponding design response modification factor, q. θ_{mod} is expressed as:

$$\theta_{\mathrm{mod}} = \frac{\theta_{\max}}{q \cdot \theta_{1,\max}} \tag{3}$$

The influence of main structural characteristics such as design behaviour factor (q), building aspect ratio (n), and panel fragmentation (m), on the inelastic deformation demands in multi-storey CLT buildings are presented in Figures 4 and 5. It is clear from Figure 4 that the correlations between global drift demands and frequency content of the ground motion is non-linear along the full range of period ratios (T_1/T_m). The period ratio, T_1/T_m, (or tuning ratio) is defined as the ratio between the structural period associated with the first vibration mode of the structure and the mean period of the ground-motion.

Figure 4a presents the variation between δ_{mod} and T_1/T_m , for specific design response modification factors (i.e. q = {2,2.5,3,4}) and for the 8-storey building with the lowest level of panel fragmentation. It is clear from this figure that non-linear evolution of drift demands is a function of the inelastic response indicated by the response modification factor, q. That is to say, higher the response modification factors correspond to higher energy dissipation capacity and lower inelastic deformation demands.

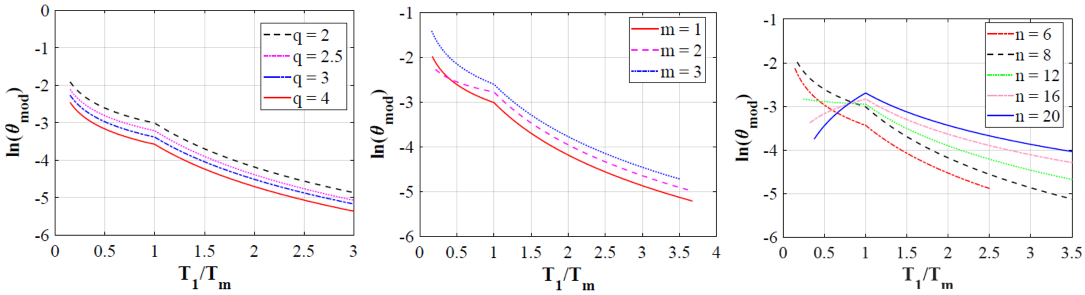
The correlation between δ_{mod} and T_1/T_m, for different levels of panel fragmentation is depicted in Figure 4b. It can be appreciated from Figure 4b that the increase in the number of vertical joinery lines per wall corresponds to larger global drift modification factor. This means that additional multiple rocking segments in modularized configurations are associated with a more flexible response. Furthermore, increasing the number of vertical connection lines from m = 1 to m = 2 leads to higher drift increments for tuning ratios bigger than 0.5, in comparison to increasing the number of vertical connection lines from m = 2 to m = 3. Thus, the reduction in stiffness brought about by panel fragmentation, in comparison with the relative benefits of enhanced energy dissipation capacity, is more important for individual panel lengths of 2.85 m or less. The relationship between δ_{mod} and T_1/T_m, for various aspect ratios (n) is shown in Figure 4c. It can be seen in Figure 4b that global drift modification factor decreases with increasing period ratio regardless of the building height in the long period range, where $T_1/T_m > 1$. Besides, it can be seen that higher global drift demands are expected in 6- and 8-storey structures along the full range of period ratios. On the other hand, the evolution of non-linear drifts demonstrates different behavioural trends in the short period range ($T_1/T_m < 1$) based on the number of storeys.



(a) *(b)* *(c)*

Figure 4. Relationship between ln(δ_{mod}*) and *T_1/T_m* for various response modification factors (a), level of panel fragmentation (b), and number of storeys (c).*

Similarly, the influence of key structural as well as ground-motion characteristics on maximum drift demands can be explored by means of Figure 5. It can be appreciated in Figure 5a and 5b, as already observed in global drift demands, the same correlations are expected for the influence of design response modification factors and level of panel fragmentation on the maximum drift demands. However, a different trend is observed for the influence of number of storeys. Figure 5c presents the variation in maximum drift modification factor for a range of building heights (n). It is clear from this figure that in the short period range $T_1/T_m < 0.5$, higher maximum drift modification factors are expected in 6- and 8-storey buildings. The main scaling features observed in global drift demands can also be seen for maximum drift demands. On the other hand, in the long period range where $T_1/T_m > 1$, higher inter-storey drift demands are observed in taller CLT buildings. This highlights that although taller CLT buildings experience smaller global inelastic demands overall, higher normalized local seismic deformation demands become more pronounced due to the influence of higher mode-effects in tall buildings.



(a) *(b)* *(c)*

Figure 5. Relationship between ln(θ_{mod}*) and T1/Tm for various response modification factors (a), level of panel fragmentation (b), and number of storeys (c).*

Seismic force demands

As highlighted before, there is no study conducted to date with a focus on the evolution of seismic force demands in multi-storey CLT buildings. Therefore, this section outlines the seismic force demands in multi-storey CLT buildings, and the influence of structural as well as ground-motion characteristics on them. Two well-known modification factors are computed to guide the study of inelastic shear demands:

- Base shear modification factor, V_{mod}, is the ratio between the maximum base shear (V_{max}), obtained from non-linear response history analysis and the product of the plasticity resistance ratio (α) and the base shear at yield (V_1). V_{mod} can be expressed as:

$$V_{mod} = \frac{V_{max}}{\alpha * V_1} \tag{4}$$

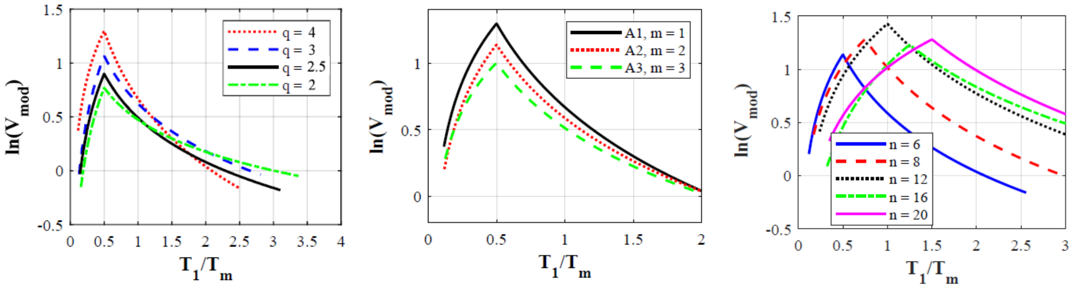
- Inter-storey shear modification factor ($V_{st,mod}$) is the ratio between the maximum inter-storey shear at the i^{th} storey of the building ($V_{i,max}$) obtained from non-linear response history analysis and the product of the plasticity resistance ratio (α) and total inter-storey shear at the i^{th} storey at the first yield ($V_{i,1}$) defined before:

$$V_{st,mod} = \frac{V_{i,max}}{\alpha * V_{i,1}} \tag{5}$$

The influence of main structural characteristics on the seismic force demands can be studied with reference to Figures 6 and 7. The relationship between V_{mod} and T_1/T_m for various response modification factors, levels of panel fragmentation, and number of storeys is presented in Figure 6 whereas similar correlations are shown for V_{stmod} in Figure 7. The evolution of base shear demands attained in multi-storey CLT buildings as a function of behaviour factor is illustrated in Figure 6a with reference to 6-storey A1 building. In this figure, different trends are seen for the short ($T_1/T_m < 0.5$) and long ($T_1/T_m > 0.5$) period ranges. The main feature of this correlation is that peak base shear ratios increase in direct proportion to the q values, or in other words, in direct proportion to the energy dissipation capacity of the building. This is reasonable as larger forces, relative to the corresponding yield values, are expected for higher ductility levels in strongly hardening structures.

The variation between the mean values of base shear demands, V_{mod}, and tuning ratios, T_1/T_m, for various levels of panel fragmentation is depicted in Figure 6b. It can be appreciated from this figure that the mean base shear modification factor decreases with increasing levels of panel modularization, suggesting that more vertical joint lines lead to lower shear demands. This reflects the enhanced energy dissipation capacity associated with multiple rocking segments coupled with the more pronounced post-yield hardening effects of long CLT panels.

Finally, the relationship between V_{mod}, and tuning ratios, T_1/T_m, for different building heights is explored in Figure 6c. This figure presents a comparison of the results of CLT building models of different heights with a single vertical joinery line, designed with a behaviour factor of q = 3. It is clear from this figure that the mean base shear modification factor increases non-linearly with increasing period ratios regardless of the building height in the short period range, peaking at peculiar period ratios depending on the building height (i.e. $T_1/T_m = \{0.5, 0.75, 1.0, 1.25, 1.5\}$). This observation indicates the importance of resonant response (at the first building mode) in the evaluation of seismic shear demands. Moreover, the largest base shear modification factors are observed in buildings of around 12 storeys which the fundamental period of the buildings is equal to the mean period of the ground motion ($T_1/T_m = 1$).

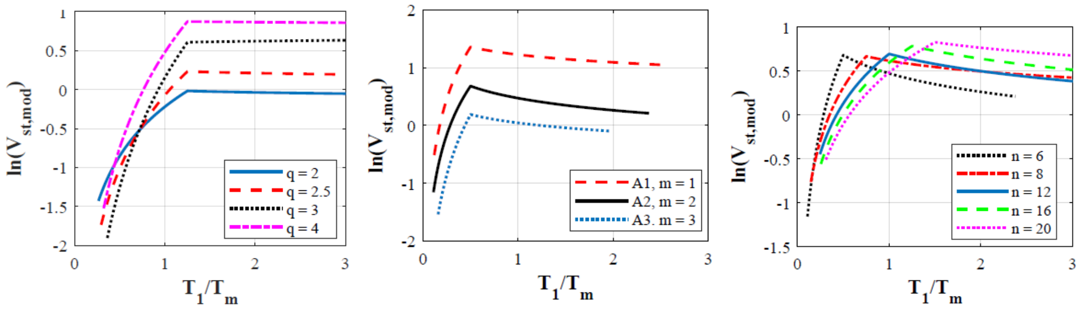


(a) *(b)* *(c)*

Figure 6. Relationship between ln(V_{mod}) and T_1/T_m for various response modification factors (a), level of panel fragmentation (b), and number of storeys (c).

Similarly, the influence of main structural parameters on inter-storey shear demands is examined with reference to Figure 7. Figure 7a displays the variation in mean inter-storey shear modification factor $V_{st,mod}$, as a function of the period ratio, in the 16-storey building for different design behaviour factors. It is clear from this figure that larger inter-storey shear demands are expected with larger design response modification factors along the full period range. As before, these tendencies are explained by the significant levels of post-yield hardening experienced by tied down CLT panels.

The variation between $V_{st,mod}$ and T_1/T_m for various joint density parameters is shown in Figure 7b. The base shear modification factor, $V_{st,mod}$ decreases with increasing number of vertical joint lines suggesting that larger number of ductile connectors corresponds to an overall increment in the energy dissipation capacity of the structure. Finally, Figure 7c presents the correlation between $V_{st,mod}$ and T_1/T_m for different building heights. It is clear from this figure that while lower shear demands are expected in the short period range for taller structures, larger shear demands will take place in taller buildings in the long period range. This is attributed to the importance of higher mode-effects in the seismic response of buildings which is more pronounced in structures with higher slenderness values.



(a) *(b)* *(c)*

Figure 7. Relationship between ln($V_{st,mod}$) and T_1/T_m for various response modification factors (a), level of panel fragmentation (b), and number of storeys (c).

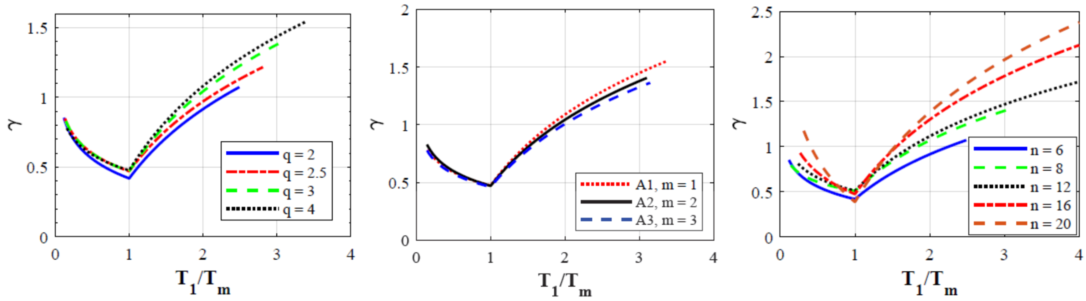
Seismic acceleration demands

As highlighted in previous research studies, multi-storey CLT buildings are prone to high floor accelerations. Besides, a realistic assessment of seismic floor acceleration demands is fundamental for the appraisal of damage to non-structural elements and building contents. Therefore, there is a need to evaluate seismic acceleration demands in multi-storey CLT buildings. An acceleration amplification factor is defined as the ratio of the maximum floor acceleration (α_{max}) along the building height obtained from non-linear response history analysis against spectral acceleration of the ground-motion corresponding to the fundamental period of the building ($S_a(T_1)$):

$$\gamma = \frac{\alpha_{\mathrm{max}}}{S_a(T_1)} \tag{6}$$

As before with the seismic drift and force demands, the influence of salient structural and ground-motion on seismic acceleration demands are analysed with reference to Figure 8. First of all, the effect of the behaviour factor, q, on γ can be seen in Figure 8a. It is clear from this figure that lower acceleration levels are expected with lower design behaviour factors when fundamental period of the buildings are higher than the mean period of the ground-motion. On the other hand, the influence of the behaviour factor is negligible for $T_1/T_m < 1$. This variation supports the previous findings with regards to the importance of higher mode effects in the long period range and the mitigation of peak floor accelerations associated with the period elongation.

On the other hand, Figure 8b summarizes the distribution of acceleration amplification factors with different wall arrangements. It is clear from this figure that the level of panel fragmentation is noticeable but less important. The mitigating effects of higher levels of non-linear response in peak accelerations are also evident from Figure 8b where lower seismic floor acceleration demands are expected with higher levels of panel fragmentation. Finally, Figure 8c provides an additional confirmation of the importance of higher mode effects by means of observations on seismic floor acceleration demands. Mean acceleration demands in multi-storey CLT buildings of different heights are compared in this figure and it is found that taller buildings tend to experience higher floor acceleration demands due to higher mode effects.



(a) *(b)* *(c)*

Figure 8. Relationship between γ and T_1/T_m for various response modification factors (a), level of panel fragmentation (b), and number of storeys (c).

Prediction Models

As a results of the extensive non-linear analyses performed and the identified tendencies, rigorous non-linear regression models for the estimation of seismic drift, force, and acceleration demands have been developed (Demirci *et al.*, 2018, Demirci *et al.*, 2019). These models are summarized in this section. Significant preliminary parametric investigations were conducted and only the best prediction models are offered herein. The corresponding regression coefficients for the prediction models are summarized in Equations 1 to 5. All regression coefficients have been tested for statistical significance. Besides, all models lead to a good representation of the underlaying behavioural tendencies and no residual heteroskedasticity could be identified. Detailed information regarding the residual plots and associated standard deviation can be found in Demirci (2019).

- Global drift modification factor, δ_{mod}

$$\ln\delta_{mod} = a + b*\beta + c*\lambda + (d + e*q)*\ln\left[\min\left(\frac{T_1}{T_m},1\right)\right] + f*\ln\left[\max\left(\frac{T_1}{T_m},1\right)\right] \tag{7}$$

Table 1. Regression coefficients for the global drift modification factor (δ_{mod}).

a	b	c	d	e	f
0.1162	0.2456	-0.0460	-2.0765	0.5015	-1.1442

- Maximum drift modification factor, θ_{mod}

$$\ln\theta_{mod} = a + b*\beta + c*\ln\lambda + (d + e*q)*\ln\left[\min\left(\frac{T_1}{T_m},1\right)\right] + f*\ln\left[\max\left(\frac{T_1}{T_m},1\right)\right] \tag{8}$$

Table 2. Regression coefficients for the maximum drift modification factor (θ_{mod}).

a	b	c	d	e	f
-4.3548	0.2421	0.5228	-1.7093	0.4027	-1.0708

- Base shear modification factor, V_{mod}

$$\ln V_{\mathrm{mod}} = a + b * \lambda + (c + d * q) * \ln\left[\min\left(\frac{T_1}{T_m}, \xi\right)\right] + e * \ln\left[\max\left(\frac{T_1}{T_m}, \xi\right)\right] + f * \beta + g * \alpha \qquad (9)$$

Table 3. Regression coefficients for the base shear modification factor (V_{mod}).

a	b	c	d	e	f	g
2.1169	0.1205	0.2150	0.1315	-0.7906	-0.2269	-0.3678

- Storey shear modification factor, $V_{st.mod}$

$$\ln V_{\mathrm{st,mod}} = a + b * \lambda + (c + d * q) * \ln\left[\min\left(\frac{T_1}{T_m}, \xi\right)\right] + e * \ln\left[\max\left(\frac{T_1}{T_m}, \xi\right)\right] + f * \beta + g * \alpha \qquad (10)$$

Table 4. Regression coefficients for the storey shear modification factor ($V_{st,mod}$).

a	b	c	d	e	f	g
3.7369	-0.1567	0.7733	0.1410	-0.0339	-0.3317	-0.5105

- Acceleration amplification factor, γ

$$\gamma = a * \beta + b * q * \ln\left[\min\left(\frac{T_1}{T_m}, 1\right)\right] + c * \ln\left[\max\left(\frac{T_1}{T_m}, 1\right)\right] \qquad (11)$$

Table 5. Regression coefficients for the acceleration amplification factor (γ).

a	b	c
0.4112	-0.0775	-0.0929

Conclusion

A detailed account of the influence of key structural parameters and ground-motion frequency content on the inelastic demands in multi-storey CLT buildings has been presented in this paper. This includes the assessment of seismic drift, force, and acceleration demands and their dependency on the ground-motion frequency content.

Higher design response modification factors (q), correspond to lower levels of seismic drift, or what is the same, a higher energy dissipation capacity of the structure, as expected. Besides, although overall lower level of inelastic deformation demands is expected in taller CLT buildings, higher localized inelastic deformations are observed in taller buildings due to higher mode effects.

Additionally, seismic force demand assessments showed that larger inter-storey shear demands are expected for larger design response modification factors (q) along the full period range. This is explained by the significant levels of post-yield hardening experienced by tied down CLT panels. Furthermore, both base and inter-storey shear modification factors decrease with increasing number of vertical joint lines highlighting the benefits of adopting a larger number of ductile connectors, hence increasing the energy dissipation capacity of the structure.

Similarly, the assessments on seismic floor acceleration demands highlighted the mitigating effects of higher levels of non-linear response in peak accelerations. In this respect, lower seismic floor acceleration demands are expected with higher levels of panel fragmentation. Moreover, the importance of higher mode effects have been further confirmed in taller, more slender buildings leading to significantly higher floor accelerations in taller CLT buildings.

Finally, a set of regression models for the estimation of seismic demands in multi-storey CLT buildings have been presented. They constitute a simple tool to identify the seismic drift, force, and acceleration demands in CLT buildings and can be incorporated as a guidance in seismic design and assessment provisions for CLT buildings.

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