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DRIFT RESPONSE OF TALL CROSS-LAMINATED TIMBER BUILDINGS UNDER REALISTIC EARTHQUAKE LOADS

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ABSTRACT

This paper examines the drift response of tall cross-laminated timber (CLT) buildings subjected to a large set of real strong ground motions. Particular focus is placed on the influence of ground-motion frequency content on the inelastic drift demands of multi-storey CLT building structures. A total of 68 CLT buildings with varying structural characteristics were modelled and subjected to a set of 1656 real acceleration records. The effect of the frequency content of ground-motion, characterised by its mean period, T_m, is found to be determinant on the inelastic deformation demands of CLT walled buildings. Furthermore, the evolution of drift demands as a function of tuning ratio reveals different trends for low and high-rise CLT buildings. Prediction models for the estimation of global and inter-storey drift response on low-, mid- and high-rise CLT buildings are developed by means of nonlinear regression analysis. Finally, a comparative study is performed with reference to Eurocode 8 equal displacement rule and recent assessment proposals is outlined.

Keywords: CLT; Dynamic response; Tall timber building; Frequency content; Mean period.

1. INTRODUCTION

The use of cross-laminated timber (CLT) in multi-storey construction has numerous advantages over conventional building materials such as steel and reinforced-concrete in terms of its minimal environmental effects, efficient structural performance, and reduced construction time and cost. In this context, multi-storey tall CLT construction is an appealing and environmentally responsible building option with a huge potential for the optimization of the land use, especially in emerging global metropolitan cities where more than 15 million people are located (Goncalves and Umakoshi, 2010). In addition, the inherent in-plane stiffness and the possibility of ductile connection design in CLT construction offers an appealing building alternative for earthquake prone areas.

The seismic response of CLT buildings has been the issue of numerous experimental and numerical studies for more than a decade. One of the most comprehensive experimental projects on the seismic behaviour of CLT construction (SOFIE project) was carried out by CNR-IVALSA. The project included 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 ground-motion records on a shaking-table (Ceccotti et al., 2006; Ceccotti et al., 2013). The shear-wall test results on various panel configurations obtained within the framework of the SOFIE project was reported by Gavric et al. (2012; 2015). In addition, other experimental research studies were conducted by Popovski and Gavric (2015), Yasumura et al. (2015), Flatscher and Schickhofer (2015), and Málaga-Chuquitaype et al. (2016) to investigate the lateral response of CLT buildings and panel assemblies. These studies have shown that well-designed

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and appropriately-assembled CLT structures are able to develop good seismic resistance and ductility under lateral loads.

A large amount of numerical research has also been carried out on the lateral response of CLT buildings. Most of the numerical research investigations have focused on the calibration of numerical models against experimental results, and on the characterisation of appropriate response modification factors for low- to mid-rise CLT buildings (Ceccotti and Sandhaas, 2010; Pozza and Scotta, 2015; Rinaldin and Fragiacomo, 2016). However, no comprehensive studies have been identified on the seismic response regarding inelastic deformation demands in multi-storey CLT buildings. Although seismic design provisions exist in North America (Gagnon and Ciprian, 2011; Karacabeyli and Douglas, 2013) and proposals for the new edition of European provisions (CEN, 2004) can be found in research studies, these guidelines tend to concentrate on strength considerations and on the recommendation of design behaviour factors (Follesa et al., 2015) rather than on the assessment of drift demands. This is despite the fact that deformations, as opposed to strength, are well correlated with earthquake damage.

It follows from the preceding discussion, that to date there is a need for a comprehensive assessment of the expected level of drift demands in multi-storey CLT buildings and the influence of ground-motion characteristics on their response. Moreover, the effect of ground-motion frequency content, which has paramount importance in seismic response assessment of buildings, has been often ignored in previous research on CLT buildings. To this end, this paper provides a detailed account of the inelastic deformation demands in tall CLT buildings and the influence of ground-motion frequency content on their response.

2. STRUCTURAL CONFIGURATION AND DESIGN DETAILS

2.1 Seismic Design of Prototypical CLT Buildings

The present section provides a detailed account of the seismic design of multi-storey CLT buildings covering a wide range of structural characteristics. As discussed earlier, there is no official guidance considering in detail the seismic design of CLT structures in Europe. However, capacity design considerations and failure mode control principles, as described in Eurocode 8 (CEN, 2004), were followed in this study.

A typical plan view and elevation corresponding to the 6-storey CLT building are illustrated in Figure 1. The same plan layout was employed for all building heights. To this end, a range of number of storeys, n, was considered (i.e. n = 6, 8, 12, 16, 20). 5-layered CLT panels with varying thicknesses between 95 and 200 mm were employed for the shear walls in both directions (x- and y-axis), whereas 5-layered 200 mm thick CLT panels were used for the roof and floor slabs in all building configurations. 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 total dead load and 30% of superimposed load.

The Eurocode Type 1 response spectrum (for high seismicity areas) was adopted for the design with soil type C conditions. Therefore, the following spectral parameters were considered: S = 1.15, T_B = 0.2 s, T_C = 0.6 s, and T_D = 2.0 s. A reference peak ground acceleration of $\alpha_{gR} = 3.0$ m/s^2 was adopted with an importance factor of $\gamma_i = 1.0$. C24 timber class from BS 25 EN 14081-1:2005 was used for all CLT panels. To obtain design material properties, characteristic values were multiplied by a strength modification factor of $k_{mod} = 1.1$ and divided by a partial safety factor of $\gamma_M = 1.0$ as suggested in Eurocode 8 (CEN, 2004) for seismic scenarios.

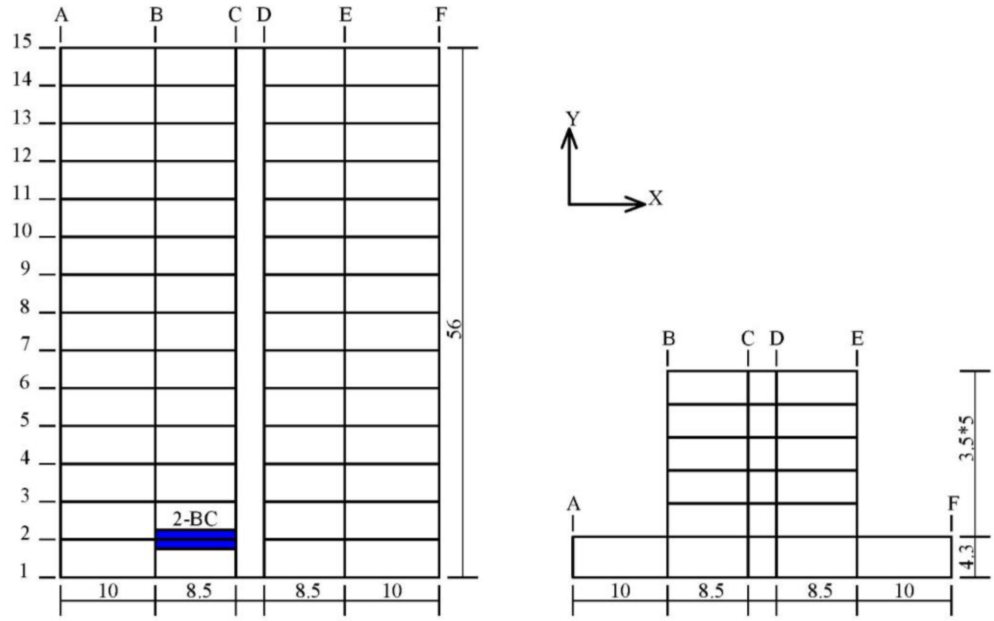


Figure 1. Plan view and the elevation of a typical 6-storey CLT building.

It is important to note that although a design response modification factor of q = 3 has been recommended by other researchers (Ceccotti, 2008; Ceccotti and Sandhaas, 2010; Sustersic et al., 2015), a range of additional q factors (i.e. q = {2, 2.5, 3, and 4}) were adopted in this study to assess the influence of different design assumptions on the inelastic response of CLT buildings. In addition, to investigate the influence of wall fragmentation that are associated with the density of vertical joint lines (m), a number of joint densities was considered, namely m = {0, 1, 2, and 3}. It should be noted that although the single panel cases (m = 0) have been considered for the 6- and 8-storey CLT buildings due to their widespread current use, this long panels are not recommendable for seismic resistant buildings due to ductility considerations (Málaga-Chuquitaype et al., 2016). The number of connectors and vertical joints were determined to comply with capacity design principles and a detailed account of the type and distribution of structural timber connections can be found elsewhere (Demirci et al., 2018; Demirci, 2018).

2.2 Structural characteristics

In order to perform a detailed parametric investigation on the drift demands in multi-storey CLT buildings, a large set of models was constructed. This set comprises 68 numerical models of CLT building cores, which main structural characteristics can be summarised as follows:

1. The fundamental period, T_1, as obtained from an elastic modal analysis, ranges from 0.3 to 1.0 s. The upper panel in Figure 2 shows the distribution of fundamental periods of the building dataset employed.

2. The building aspect ratio, λ, defined as the quotient between the height and the width of the building façade ranges from 2.56 to 8.33. The distribution of building aspect ratios is depicted in the middle panel of Figure 2.

3. The joint density parameter, β, is defined by Pozza and Trutalli (2017) as the ratio of the joint lines (P_0) on the CLT wall to the perimeter of the wall panel (P). The distribution of joint density parameters is indicated in the lower panel of Figure 2 and ranges from 1.70 to 3.36. These values were then used to determine a corresponding response modification factor following the proposal of Pozza and Trutalli (2017) for comparison with the present study. A more detailed account of the computation of joint density parameters and proposed response modification factors can be found in Demirci et al. (2018).

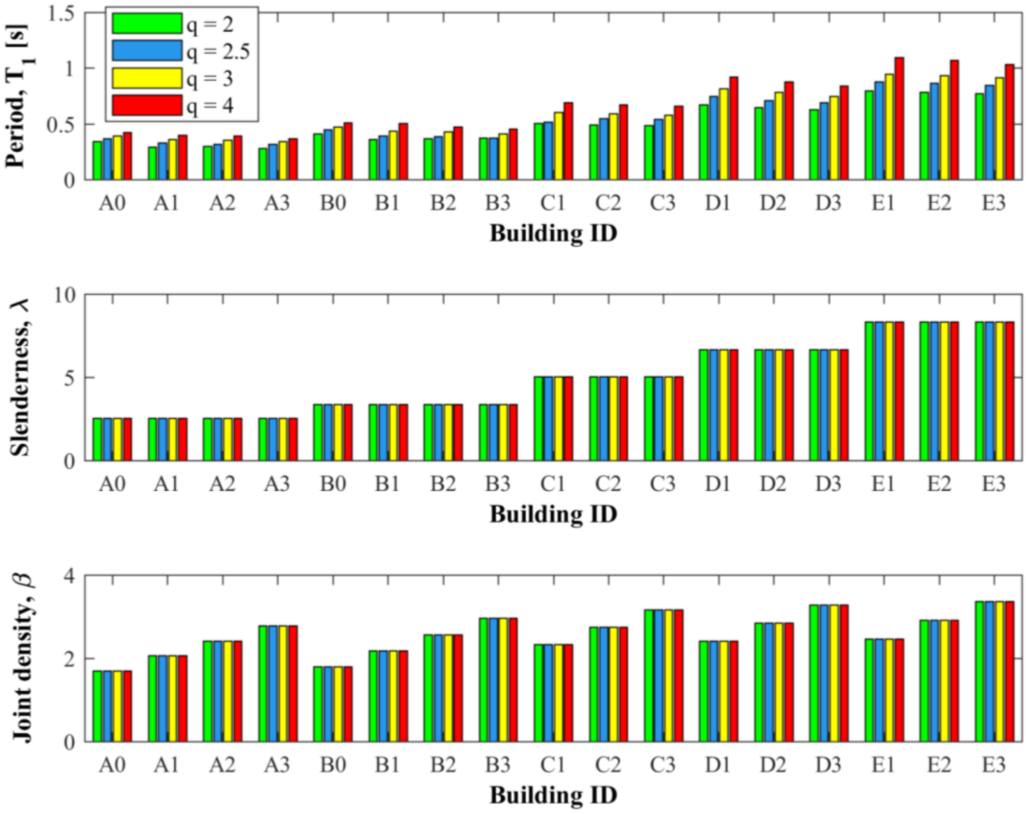


Figure 2. Distribution of structural characteristics of the considered building configurations in the study.

3. NUMERICAL MODELLING

This section presents the numerical modelling approach adopted in order to perform nonlinear static and dynamic analyses. The models are based on the database of CLT structures designed and described in the preceding section. The model validation is also presented below.

3.1 Modelling Assumptions

Numerical models of representative prototypical CLT buildings were constructed in the open-source Finite Element framework OpenSees (McKenna, 2011). The CLT core section under consideration, 2-BC, is shown in Figure 1 whereas the adopted numerical modelling approach is depicted in Figure 3. Linear elastic 4-node Quad elements were used to simulate CLT panels. Nonlinear zero-length and two-node link elements were employed to simulate structural joints. Isotropic material properties were assumed for the CLT panels based on preliminary numerical studies on the influence of orthotropic material modelling at various levels of vertical loading (Demirci, 2018). It is important to note that Quad elements in OpenSees (McKenna, 2011) are based on 2-degree-of-freedom per-node idealization and only allows a linear geometric transformation. Therefore, a leaning column was modelled using beam-column elements with 3-degree-of-freedom per-node to account for P-delta second order effects. The leaning column was attached to the CLT core section with equal-degree-of-freedom multi-point constraints (EdofMP) to simulate the rigid diaphragm connection between the two model domains. The corresponding vertical loads were applied to the leaning column at each floor level. Tri-linear hysteretic material properties available in OpenSees (McKenna, 2011) were assigned to structural joints, namely shear brackets, tie-down connectors, and structural joints between adjacent wall panels. All connection elements were calibrated to experimental results (Gavric et al., 2015) and analytical calculations. To this end, experimentally calibrated degradation parameters in the range of 0.2 and 0.8 were found to be representative of closest estimations to the hysteretic behaviour of structural joints (Demirci et al., 2018; Demirci, 2018).

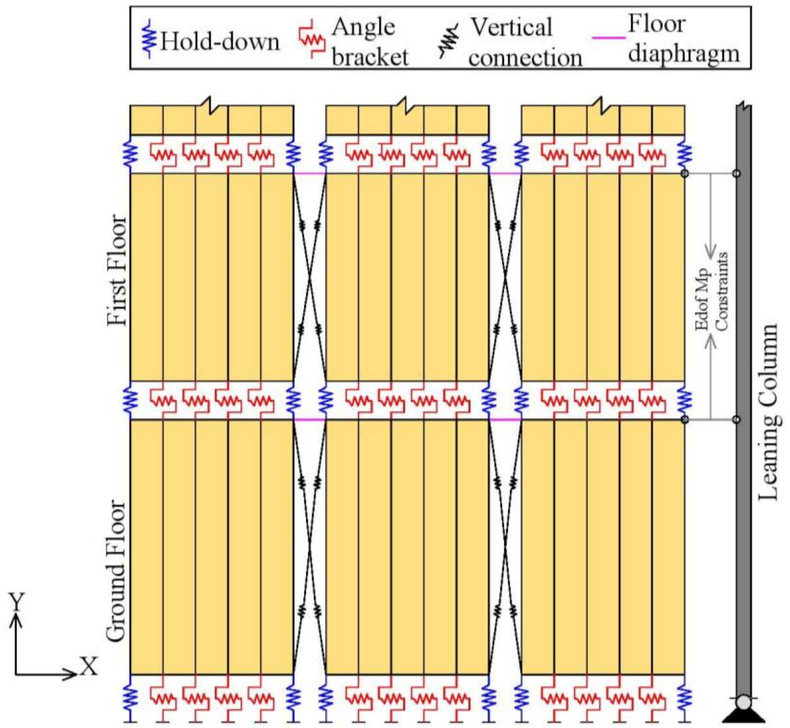


Figure 3. Numerical modelling approach for CLT core section.

3.2 Model Validation

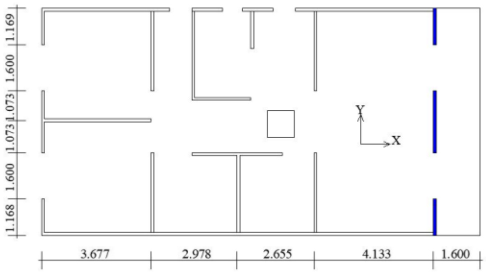
The numerical modelling approach described above was extensively validated against available test results. A detailed account for the validation can be found in Demirci (2018) and Demirci et al. (2018). As an example, Figures 5a and 5b present comparisons between the shaking table results of a 7-storey CLT building (SOFIE building) reported in Ceccotti et al. (2006; 2013), and Rinaldin and Fragiacomo (2015) with the numerical estimations of the present work. While a 3D ground excitation had been applied during the full-scale shaking-table test, only y- direction was considered in this study. However, a good agreement was found between the outcomes from the numerical model adopted in this study and the experimental results. Figure 5 presents the numerical-experimental comparison of displacement and acceleration histories correspond to the top floor of the building subjected to Kobe JMA 1995 earthquake. Other dynamic properties of the numerical model of the SOFIE building were also found in agreement with experimentally obtained values. Table 1 summarizes one of such comparisons in terms of fundamental periods.

Table 1. Fundamental period comparison of SOFIE building.

Fundamental period	Shake table measurement	OpenSees model	Error [%]
T_1	0.427	0.463	8.43
T_2	0.301	0.313	3.98

4. EARTHQUAKE RECORDS AND FREQUENCY CONTENT

From 51 seismic events, a total of 1656 real ground-motion records with magnitudes, M_w, ranging from 5.61 to 7.9 and with average PGA of 1g were employed during the dynamic analyses of buildings. The acceleration time histories were obtained using PEER-NGA database involving different site classes and fault types. Detailed information regarding selected earthquake records can be found in Demirci et al. (2018) and Hancock et al. (2008).



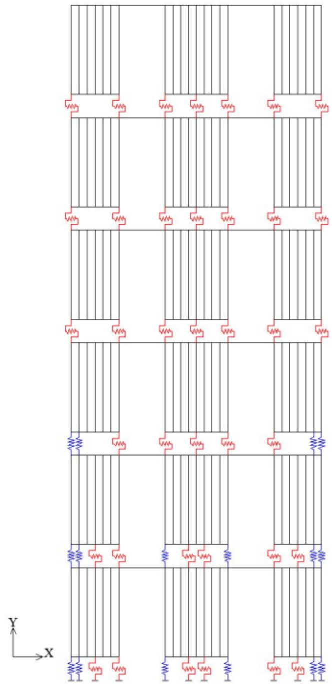


Figure 4. Plan view and numerical model schematization of 7-storey SOFIE building

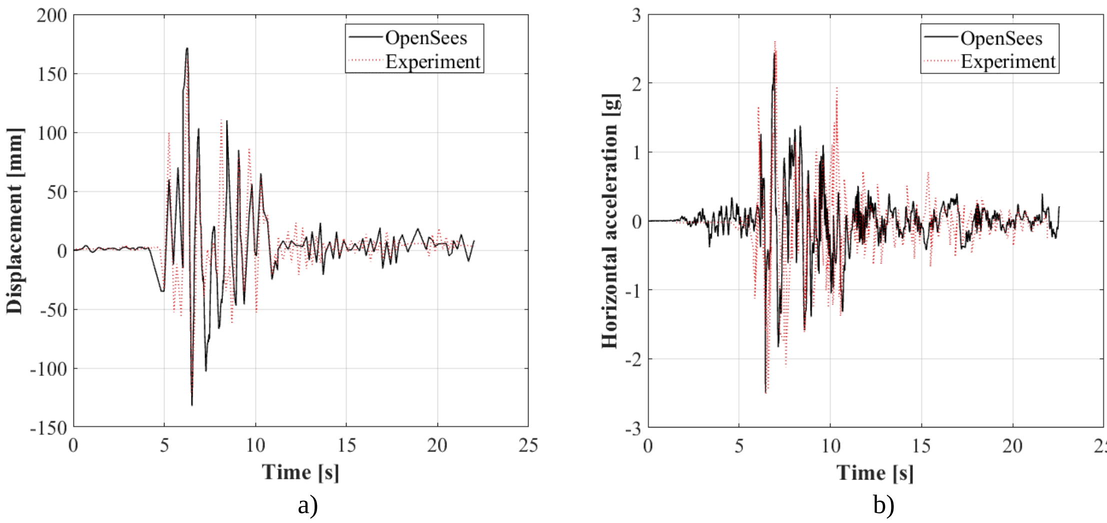


Figure 5. Numerical – experimental comparison of top floor displacement and floor acceleration of SOFIE building.

The mean period (T_m) of the ground-motion was selected as the index to characterize frequency content of ground-motion (Rathje et al., 1998; 2004) based on previous studies (Málaga-Chuquitaype et al., 2012; 2015). The mean period of the ground-motion can be determined as follows:

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

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

5. SEISMIC DRIFT DEMAND ASSESSMENT

A large number of nonlinear response history analyses (≈112,608) were performed by means of an array function on the high-performance research computing facility at Imperial College London (HPC, 2017).

The results were then post-processed to obtain the maximum displacements (Δ_{max}) at the roof level and maximum drifts (θ_{max}) at each storey level. Finally, these values were used to calculate the global and maximum drift modification factors as described below:

- The Global Drift Modification Factor, δ_{mod}, is the ratio between the maximum roof displacement Δ_{max} (obtained from nonlinear response history analysis) and the product of the roof yield displacement $\Delta_{1,roof}$ (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:

$$\delta_{mod} = \frac{\Delta_{max}}{q \cdot \Delta_{1,roof}} \quad (2)$$

- The Maximum Drift Modification Factor, θ_{mod}, is the ratio between the maximum roof drift θ_{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:

$$\theta_{mod} = \frac{\theta_{max}}{q \cdot \theta_{1,max}} \quad (3)$$

A statistical study was conducted to assess the influence of numerous structural characteristics on global and maximum drift demands in multi-storey CLT structures. In addition, predictive models were developed to investigate the trends between the drift demands attained by CLT walled structures and the ground-motion frequency content.

5.1 Global Drift Modification Factor

The variations in δ_{mod} as a function of the tuning ratio (T_1/T_m) and for a design response modification factor of q = 3 are presented in Figure 6 for the 8-storey B1 (m = 1) and 20-storey E3 (m = 3) building configurations. The tuning ratio is calculated as the ratio between the first fundamental period of the structure (T_1) and the mean period of the ground motion (T_m). In Figure 6, the open (o) and asterisk (*) symbols denote mean and median values of drift modification factor within bins, respectively. The shaded region illustrates the 95% confidence interval in the estimate of mean logarithmic drifts. The dotted lines indicate the ± one standard deviation of the mean logarithmic global drift modification factor ($\ln\delta_{mod}$) shown by the solid line for the best prediction model described in the next section. In addition, the dash-dot lines in these figures specify the value of $\delta_{mod} = 0$ corresponding to the equal displacement rule usually adopted in European design provisions while dashed lines indicate the correlation obtained employing the equation proposed by Pozza and Trutalli (2017). A more complete account of this can be found in Demirci et al., (2018).

It is clear from Figure 6 that the relationship between inelastic deformation demands and the frequency content of the ground-motion is nonlinear along the full range of tuning ratios for both building heights. The nonlinear evolution of drift demands is important since linear scaling is usually adopted for drift estimations particularly in long-period buildings, based on the results for generalized single-degree-of-freedom systems (Ruiz-Garcia and Miranda, 2003; Málaga-Chuquitaype and Elghazouli, 2012).

Besides, the variation in δ_{mod} on the frequency content of the ground-motion follows different trends for 8- and 20-storey buildings. In the case of 8-storey B1 building configuration (Figure 6a), an increase in the global drift modification factor is appreciated for decreasing tuning ratios in the short period range, $T_1/T_m < 1$. This can be attributed to the period elongation that the secant structural period approaches to the mean period (T_m) of the ground-motion causing higher drift demands in this range. However, for the long period range, $T_1/T_m > 1$, lower drift estimations are observable because of increasing divergence between the fundamental period of the structure (T_1) and the mean period (T_m) of the ground-motion.

On the other hand, a different correlation is observed for the 20-storey E3 building which can be seen in Figure 6b. In this case, δ_{mod} increases with increasing tuning ratios, T_1/T_m, in the short period range (i.e. $T_1/T_m < 1$) and reaches its peak at $T_1/T_m = 1$. This is explained by a distinctive resonant response ($T_1 = T_m$) are expected in taller buildings due to their overall lower inelastic deformation demands compared to shorter buildings where earthquake, rather than wind, is the governing design consideration.

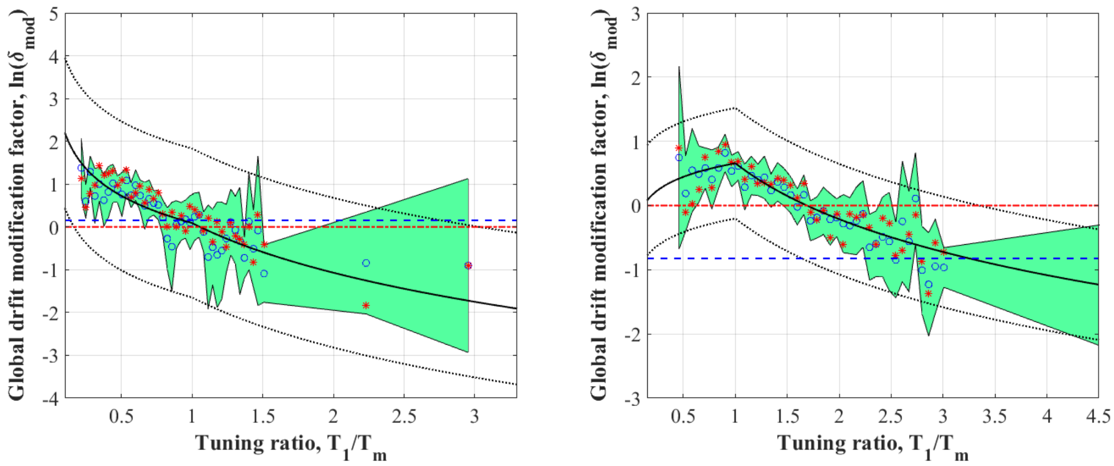
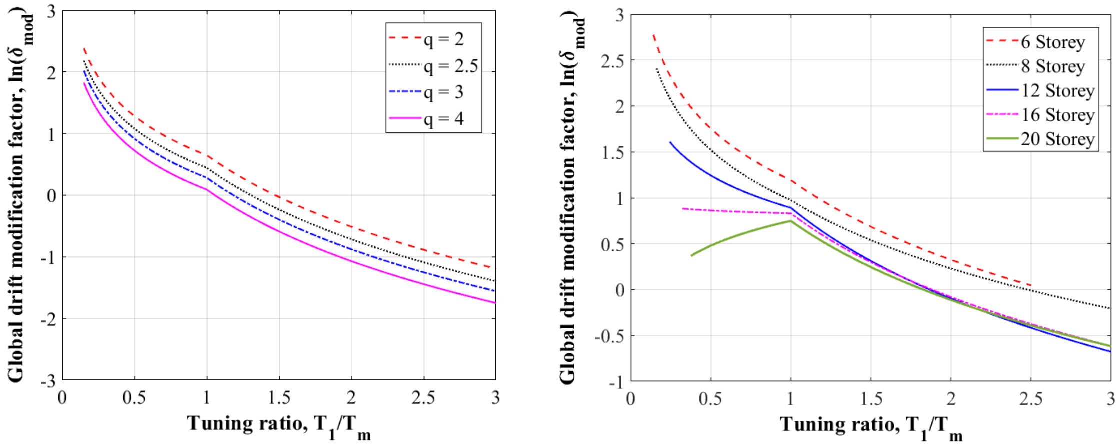


Figure 6. Mean global drift modification factor (δ_{mod}) against tuning ratio (T_1/T_m) for 8-storey B1 building (a) and 20-storey E3 building (b).

The influence of main structural characteristics such as design response modification factor (q), building aspect ratio (n), and panel fragmentation (m) on the inelastic deformation demands in CLT buildings were also investigated. Figure 7a shows the correlation between δ_{mod} and T_1/T_m for different response modification factors of q = {2, 2.5, 3, 4} for the 8-storey B1 building configuration with the lowest level of panel fragmentation (m = 1). Similar results were attained for the other building configurations examined. It can be seen in this figure that the nonlinear evolution of drift demands is a function of the inelastic response indicated herein by the response modification factor, q. In other words, the higher response modification factors correspond to lower inelastic deformation demands and higher energy dissipation capacity of the structure.

Similarly, Figure 7b presents the variation between the global drift modification factor (δ_{mod}) and the tuning ratio (T_1/T_m) for various building storeys (n). More precisely, the inelastic deformation demands of CLT structures of different heights designed for the response modification factor of q = 3 while keeping the density of joints same (i.e. m = 1) were compared. It is clear from this figure that the global drift modification factor decreases with increasing tuning ratio regardless of the building height in the long-period range ($T_1/T_m > 1$). It can also be appreciated that higher global drift demands are expected in 6- and 8-storey buildings along the full range of tuning ratios as previously stated. On the other hand, different behavioural trends in the evolution of drifts with tuning ratio in the short-period ($T_1/T_m < 1$) range was observed depending on the number of storeys. The global drift modification factor decreases with increasing tuning ratios in 6-, 8-, and 12-storey buildings while this relation is negligible for the 16-storey building and completely-opposite in the case of 20-storey building. This highlights the significance of the inelastic deformations in low- and mid-rise CLT structures.

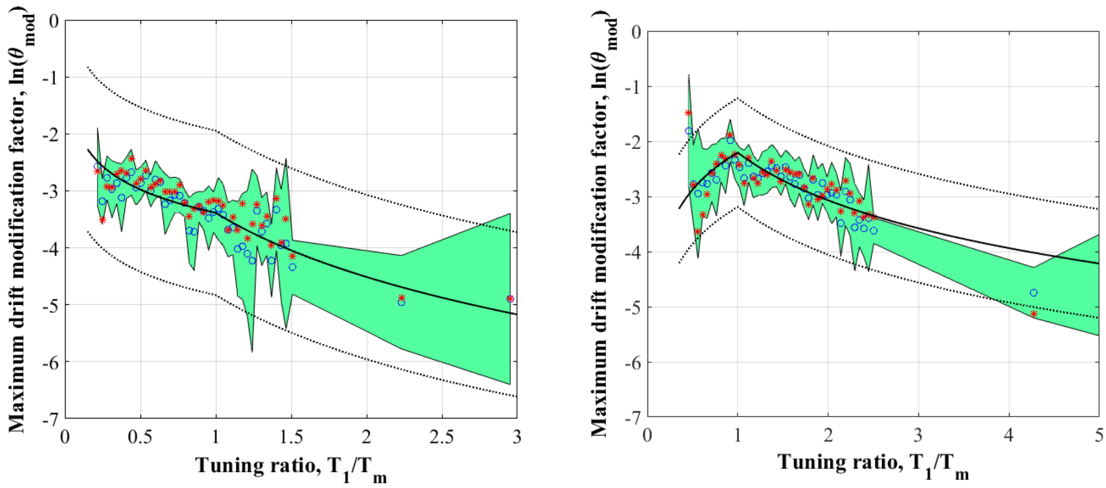


a) b)

Figure 7. Relationship between mean global drift modification factor (δ_{mod}) and tuning ratio (T_1/T_m) for different design behaviour factors (a), and building aspect ratios (b).

5.2 Maximum Drift Modification Factor

The same procedure, described above for δ_{mod}, was followed to explore the variation in maximum drift modification factor, θ_{mod} against tuning ratio (T_1/T_m) for various structural characteristics. As before, Figure 8a shows the variation in θ_{mod} for the 8-storey B1 building configuration (m = 1) as a function of the tuning ratio (T_1/T_m) and for a design behaviour factor of q = 3, whereas the variation in θ_{mod} for the 20-storey E3 building configuration (m = 3) is illustrated in Figure 8b. It is clear from these figures that general nonlinear scaling features observed for the mean global drift modification factor (δ_{mod}) are also seen for the mean maximum drift modification factor (θ_{mod}).



a) b)

Figure 8. Mean maximum drift modification factor (θ_{mod}) against tuning ratio (T_1/T_m) for 8-storey B1 building (a) and 20-storey E3 building (b).

A sample of the effect of main structural characteristics on the maximum drift modification factor (θ_{mod}) is presented in Figure 9. In these figures, the relationship between mean maximum drifts and tuning ratios (T_1/T_m) for various building heights (n) and level of panel fragmentation (m) is shown. It is clear from Figure 9a that the main scaling features observed for the global drift modification factor (δ_{mod}) are also seen for the maximum drift modification factor (θ_{mod}) in the short-period range, $T_1/T_m < 1$, where higher inter-storey drifts are expected for 6- and 8-storey CLT buildings. On the other hand, a different tendency is identifiable in the long-period range ($T_1/T_m > 1$). More precisely, taller buildings tend to have higher inter-storey drifts than shorter buildings in this range. This dependency explains that higher localized drift demands are expected in taller buildings due to the influence of higher mode effects in the long-period range, $T_1/T_m > 1$.

Finally, the correlation between maximum drift modification factor, θ_{mod} and tuning ratio, T_1/T_m for different joint density parameters (β) or level of panel modularization is depicted in Figure 9b. In this figure, the dependency on β is presented for the 8-storey B0, B1, B2, and B3 building configurations (i.e. m = {0, 1, 2, 3}). It can be appreciated from this figure that the mean maximum drift modification factor (θ_{mod}) increases for increasing number of panels per wall. This is attributed to the overall stiffness reduction effect of additional vertical connections between adjacent wall panels. These results are also consistent with the observations for global drifts (δ_{mod}) as noted in the above section.

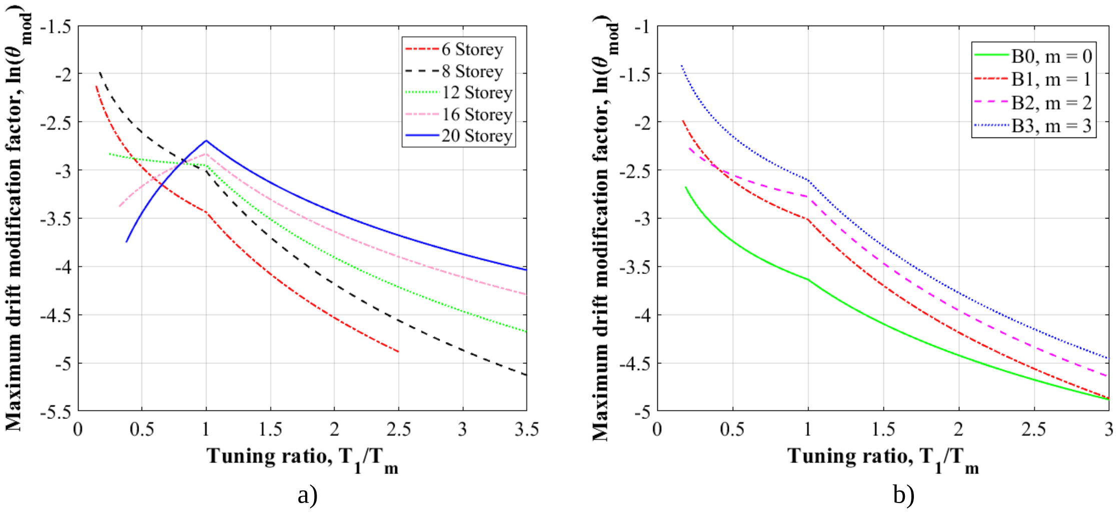


Figure 9. Relationship between mean maximum drift modification factor (θ_{mod}) and tuning ratio (T_1/T_m) for different building aspect ratios (a), and joint density parameters (b).

6. PREDICTIVE MODELS

Based on the extensive number of analyses, nonlinear regression models are developed for the estimation of global and maximum drift modification factors. Standard regression procedures were followed to identify the most appropriate expressions presented below. A discussion of such standard procedures is outside the scope and objective of the present study.

6.1 Global Drift Modification Factor

The following prediction model is proposed after a full consideration of parameters which have the greatest influence on the global drift modification factor, as discussed earlier.

$$\ln \delta_{mod} = a + b * \beta + c * \lambda + (d + e * q) * \ln\left[\min\left(\frac{T_1}{T_m}, 1\right)\right] + f * \ln\left[\max\left(\frac{T_1}{T_m}, 1\right)\right] \tag{4}$$

where δ_{mod} is the global drift modification factor, β is joint density parameter, λ is the building aspect ratio, q is the design behaviour factor, T_1/T_m is the period ratio, and a, b, c, d, e, and f are regression coefficients. To this end, the regression coefficients obtained through a nonlinear least squares algorithm are presented in Table 2. All terms of Equation 4, and their corresponding coefficients, were found to be statistically significant. A detailed account of the standard regression checks can be found in Demirci (2018) including conventional verifications of lognormality and statistical significance.

Table 2. Regression coefficients for the global drift modification factor (δ_{mod}).

a	b	c	d	e	f
0.1161	0.2456	-0.046	-2.0765	0.5015	-1.1442

6.2 Maximum Drift Modification Factor

The functional form employed for the global drift modification factor is also applied herein for the maximum drift modification factor and all statistically significant regression terms are shown in Table 3.

$$\ln \theta_{mod} = a + b * \beta + c * \lambda + (d + e * q) * \ln\left[\min\left(\frac{T_1}{T_m}, 1\right)\right] + f * \ln\left[\max\left(\frac{T_1}{T_m}, 1\right)\right] \quad (5)$$

Table 3. Regression coefficients for the maximum drift modification factor (θ_{mod}).

a	b	c	d	e	f
-4.3041	0.3821	0.0837	-1.8172	0.5028	-1.1043

Figures 6 and 8 evidence a good fit of regression models associated with Equations 4 and 5 for the global and maximum drift modification factors, respectively. These simple conventional yet rigorous models can capture well the scaling of deformation demands in multi-storey CLT buildings and can readily be incorporated in current design and assessment procedures.

7. CONCLUSIONS

This paper has presented a detailed account of the influence of main structural characteristics and ground-motion frequency content on the inelastic deformation demands in multi-storey CLT buildings. To this end, the assessment of global and maximum drift modification factors was outlined and their dependency on the frequency content of the ground-motion along the full range of tuning ratios were identified. A nonlinear scaling of the drift estimation that are peculiar to CLT structures was observed while linear scaling is usually assumed in practice. Besides, different prediction trends were found for low- and high-rise CLT buildings. Taller buildings can experience a distinct resonant response attributed to their overall lower level of inelastic deformation demands. On the other hand, relatively higher localized deformations are expected in taller CLT buildings due to higher mode effects.

A comparison can be made between the proposed prediction models and recent assessment procedures like the one recently put forward by Pozza and Trutalli (2017). Such recent proposals consider the level of panel fragmentation but they neglect the period dependency in the assessment of inelastic deformation demands in CLT buildings. This study has found that this assumption may lead to very misleading estimations outside very limited period ranges ($T_1/T_m \approx 1$ for B1 (n = 8) building and $T_1/T_m \approx 1.6$ for E3 (n = 20) building). For instance, 80% overestimations when the ground-motion has a strong short period and non-conservative results up to 70% for long period earthquakes were observed in the case of 8-storey B1 building. On the other hand, the most un-conservative results were obtained in the range of $T_1/T_m = 1$, even higher than the Eurocode estimation in the 20-storey E3 building. These results highlight that there is a need for an explicit consideration of ground-motion characteristics in the estimation of inelastic deformation demands in tall CLT buildings.

Finally, a set of regression models for the prediction of mean global drift and maximum drift modification factors have been developed. These models constitute a simple tool for the estimation of deformation demands in CLT buildings and can be readily incorporated in current design and assessment procedures.

8. ACKNOWLEDGMENTS

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