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SEISMIC BEHAVIOUR AND DESIGN OF TALL CROSS-LAMINATED TIMBER BUILDINGS

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Abstract

With half of the world's megacities located in seismic regions, there is an urgent need for tall structures that can withstand the large demands imposed by earthquakes while minimizing the environmental costs associated with their construction and maintenance. In this context, tall timber construction has the potential to enable an efficient use of urban space at the lowest possible environmental cost. In particular, Cross-Laminated Timber (CLT) panels have many advantages over other traditional structural systems such as low or negative embodied carbon, high strength-to-weight ratio and high degree of prefabrication. This study investigates the seismic design and response of high-rise buildings made of Cross-Laminated Timber (CLT). To this end, the design of an 8-storey building is presented in detail. The design follows the principles of Eurocode 8 when applicable and is informed by recent research findings when required. A detailed numerical model of the building is developed in the OpenSees Finite Element Framework with due account for geometric and material nonlinearities. The model employs 4-noded quad elements for the CLT wall panels, while two-node link and zero-length elements are used to model the non-linear response of the shear brackets, hold-downs, lap-splice connections and other contact constraints. The model is validated against available experimental data on CLT wall panels and a good agreement is obtained. Subsequently, the results of a series of static and dynamic non-linear analyses are presented and discussed with particular emphasis on ductility and strength demands.

Keywords: CLT buildings, Seismic design, tall timber building, mean period

1. Introduction

More than half of the world's population lived in cities in 2014 and it is expected that over 70% of it will be living in cities by 2050 [1]. In this context, high-rise building construction with timber has the potential to meet the urgent needs for urban buildings of the 21st century while minimizing the associated environmental effects. Today's high-rise buildings are built employing structural steel or reinforced concrete which are materials associated with high-energy consumption and significant greenhouse gas emissions throughout their manufacturing, transportation and building processes. Together, structural steel and concrete are the source of approximately 10% of total greenhouse gas emissions from the building industry [2]. In turn, buildings and transportation systems create two-thirds of the total global carbon emissions. By contrast, one cubic meter of wood can store one tonne of carbon dioxide [2] which can lead to structures with zero or negative carbon footprint.

Mid-rise construction of 4- to 6-storey timber buildings is possible by means of light-framed construction [3]; however, taller buildings of 8 to 20-storeys require an alternative configuration. To this end, massive panels of Cross-Laminated Timber (CLT) can be employed as lateral resisting elements in taller structures. CLT is an engineered timber product that is manufactured using fast growing and industrially dried spruce planks, assembled at right angles on top of each other and glued together to constitute massive solid wood panels. CLT panels are generally made of 3, 5 and 7 layers with standard panel thicknesses ranging from 57 to 320 mm (which can be fabricated up to 500 mm if required). CLT panelised construction offers a numerous advantage over traditional reinforced concrete and steel structures including ease of prefabrication and increased speed of construction. Figure 1 depicts the concept of CLT fabrication and construction.

Figure 1: Cross laminated timber assembly [4], 5-layered CLT panel [5] and a tall building made of CLT bearing walls [6]

Multi-storey CLT construction is very efficient in resisting gravity loads because CLT panels behave as solid load-bearing pieces with optimal weight-to-strength ratios. However, the dynamic response of multi-storey CLT buildings including their seismic design has not yet been fully established [7, 8]. This lack of knowledge is reflected in the dearth of codified guidance regarding their seismic design and assessment and has limited the potential of tall CLT construction in seismic areas. In fact, current multi-storey CLT buildings are mainly located in low to moderate seismic activity in Europe and Australia as summarized in Table 1. This table shows multi-storey timber buildings in CLT completed as of 2016. Nonetheless, well-designed and well-constructed timber structures can have a good earthquake response under earthquakes as demonstrated by recent shake table tests and numerical studies [8, 9]. Yet, considerable uplift forces can be experienced on ground floor hold-down connectors due to uplift and overturning seismic moments. Additionally, experimental tests have shown that when multi-storey CLT buildings are subjected to strong ground motions, they might be prone to high floor accelerations at top floor levels [8, 9, and 10].

One of the first experimental projects on the lateral response of CLT was carried out by Dujic et al. [11]. 15 CLT panels with different connection details and vertical load levels were tested under monotonic and cyclic loading. The authors concluded that the load-bearing capacity of CLT wall panels is limited by the stiffness of connections and local wood failures are possible. In addition, increasing vertical load levels have advantageous impact on the lateral resistance of CLT wall panels, especially when the connections do not have significant strength. Similarly, Popovski et al. [12] conducted numerous tests on CLT wall panel combinations with different aspect ratios and openings were examined as well as various metal connectors and connector configurations. The authors concluded that angle bracket and hold-down connectors can provide a good level of global ductility to the system, and therefore could be used in CLT building construction in seismically prone areas.

On the other hand, the SOFIE project is one of the most comprehensive research efforts on the seismic behaviour of CLT systems carried to date which included CLT shear wall panel testing as well as full-scale shake-table experimentation on 3- and 7-storey CLT buildings [8, 10]. Experimental results of this project on different connections and 20 different panel configurations have been summarized by Gavric et al. [13]. Besides, in-plane connections with screws and connections between perpendicular CLT wall panels were also examined. Based on the test results, an average value of over strength factor of 1.74 was recommended. A full-scale 7-storey CLT building was also tested on a shake table [8]. The 7-storey CLT building was designed following EC8 [14] design provisions with a response modification factor (q) of 3. During the shake table tests, the building was subjected to 10 ground-motion records with increasing intensities. Damage was observed at the hold-down connectors due to high overturning demands causing considerable uplift on tension connectors at lower levels. Nevertheless, no residual plastic deformation was experienced and the 7-storey CLT building remained stable throughout. Nonetheless, high floor accelerations of around 3.8 g were measured at the upper levels of the 7-storey CLT building. More recently, Málaga-Chuquitaype et al. [15] carried out a series of cyclic test on CLT panel of different connector configuration and found that the level of vertical loads significantly influences the local ductility of the connectors potentially leading to undesired brittle failure modes. Similarly, Popovski et al.

[16] performed a series of experiments on the in-plane response of CLT under various boundary conditions and geometric configurations. These tests were complemented with a bi-axial monotonic test on a large scale 2-storey building specimen and with subsequent comparative analytical studies on the determination of the lateral strength of CLT panels [17]. The authors concluded that methods which accounted for the sliding-uplift interaction in the connectors proved to give more consistent results compared with current design approaches that do not take into consideration the uplift resistance of brackets.

Table 1: Completed multi-storey CLT buildings

Project	Location	Storey	Height	Units	Completion
Statdhaus	London, UK	9	30	29	2009
Limnologen Project	Vaxjo, Sweeden	8	-	134	2009
Bridport House	London, UK	8	25	41	2011
Holz8	Bad Aibling, Germany	8	25	15	2011
Forte	Melbourne, Australia	10	32.2	23	2012
Cenni di Cambiamento	Milan, Italy	9	27	124	2013

This study investigates the seismic design and response of high-rise buildings made of Cross-Laminated Timber (CLT). To this end, the design of an 8-storey building is presented. The design was carried out following the principles of Eurocode 8 when applicable and making use of recent research findings when needed. A detailed numerical model of the building is developed in the OpenSees [18] Finite Element Framework with due account for geometric and material nonlinearities. The model employs 4-noded quad elements for the CLT wall panels, while two-node link and zero-length elements are used to model the non-linear response of the connections and various contact constraints. The model is validated against available experimental data on CLT wall panels subjected to cyclic loading and a good agreement is obtained. Subsequently, the results of a series of static and dynamic non-linear analyses are presented and discussed.

2. Seismic Design of Multi-Storey CLT Buildings

The present section provides a detailed account of the design of an 8-storey building made of CLT. The 8-storey building under consideration corresponds to that detailed in [19]. It represents a hotel, with a single storey podium housing, the public spaces of the hotel, surmounted by a seven-storey tower block, comprising a central corridor with bedrooms on each side. Figure 2 presents the dimensions and elevation of the 8-storey CLT building.

The structural wall thickness was chosen as 128 mm for all levels while a floor panel thickness of 200 mm was assumed. Taking into account all the finishing equipment and insulations, the total dead load of the 8-storey building was calculated as 45,124.88 kN. A superimposed load of 26,180.00 kN was estimated in accordance with the specifications for residential buildings (Service Class A). Accordingly, the seismic mass and corresponding building weight are calculated with the combination of dead load and 30% of live load. Table 2 summarizes the calculation of the seismic mass.

The Eurocode Type 1 Response spectrum (for areas of high seismicity) was used together with soil type C conditions. Therefore, from Table 3.2 in EC8 [14], the spectral parameters are:

$S = 1.15$, $T_B = 0.2$ 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 assumed while an importance factor equal to $\gamma_i = 1.0$ was considered. Besides, a behaviour factor of q = 3 was selected following the recommendations of recent research [9, 20]. Finally, the seismic base shear, F_b, for each horizontal direction in which the building analysed was determined for an estimated natural period, T_1, of 0.62 s.

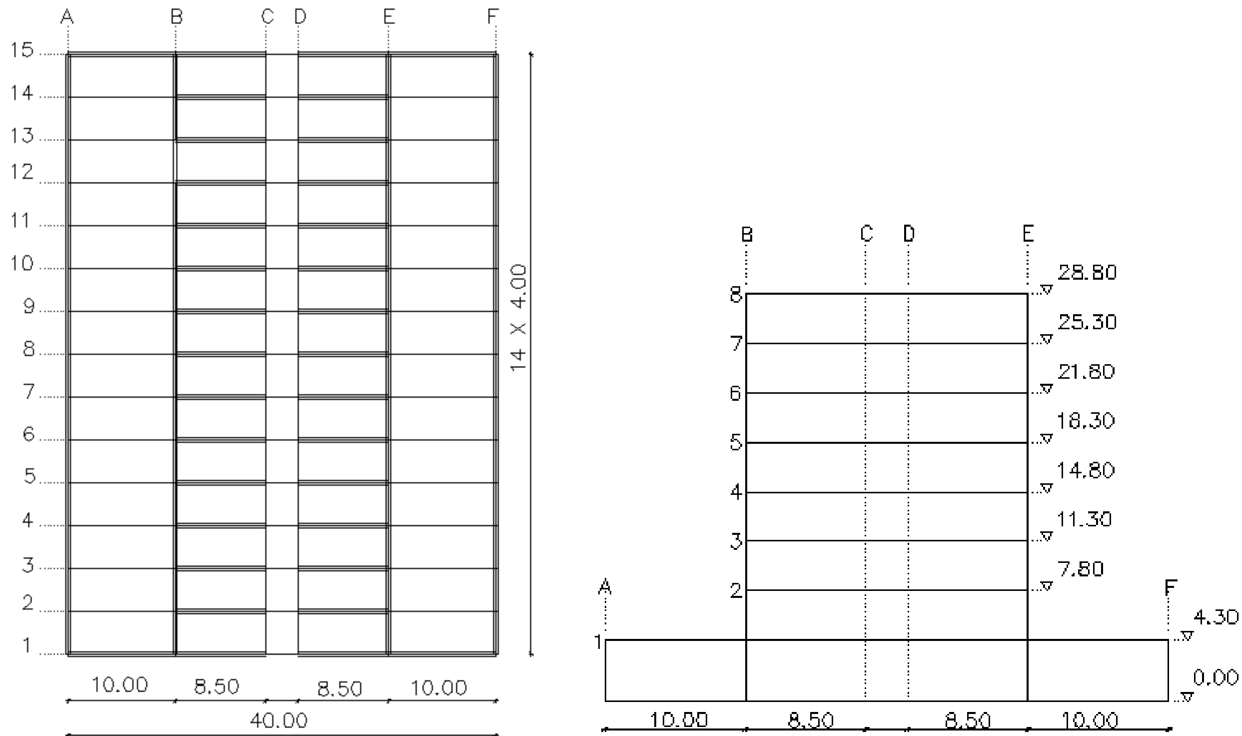


Figure 2: Plan view and the elevation of the 8-storey building

Table 2: Seismic mass calculation

Level	Dead load [kN]	Imposed load [kN]	Load combination [kN]	Mass [ton]
Ground	8878.38	7260	11,056.38	1127.44
2	5313.45	2780	6147.45	626.86
3	5313.45	2780	6147.45	626.86
4	5313.45	2780	6147.45	626.86
5	5313.45	2780	6147.45	626.86
6	5313.45	2780	6147.45	626.86
7	5313.45	2780	6147.45	626.86
8	4365.80	2240	5037.80	513.71
Total	45124.88	26180.00	52978.88	5402.31

Under these conditions, the seismic base shear is; F_b = 2.78 * 5402.31 *0.85 = 12765.66 kN. This lateral load was distributed along the height of the building using the corresponding expression in section 4.3.3.2.3(3) of EC8 [14]. Table 3 presents the distribution of lateral forces at each floor level. Likewise, since the 8-storey building is perfectly symmetrical, an accidental eccentric of 5% was considered. Table 4 provides a summary of loads carried by load bearing wall in x direction at each level of the building.

In order to be able to provide an energy dissipation level consistent with the behaviour factor (q) adopted, the length of the panels was restricted to 2.95 meters. Therefore, three panels were employed per each 8.5-metre lateral resisting wall. With this configuration, the overturning moments and shear action per panel can be determined by establishing equilibrium at the local panel level and the number and type of connector can be determined. Table 5 summarizes the type and number of metal connectors utilized. All other elements were capacity-design with an over-strength factor of 1.6 [13] in order to warrant an adequate ductile failure mechanism.

Table 3: Lateral load distribution

Floor	Height above ground (z_i) [m]	Mass (m_i) [t]	z_i * m_i [t*m]	Force (F_i) [kN]	Moment (F_i*z_i) [kN*m]
1	4.3	1127.44	4847.99	755.74	3429.68
2	7.8	626.86	4889.51	762.22	5945.32
3	11.3	626.86	7083.52	1104.23	12477.80
4	14.8	626.86	9277.53	1446.25	21404.50
5	18.3	626.86	11471.54	1788.27	32725.34
6	21.8	626.86	13665.55	2130.29	46440.32
7	25.3	626.86	15859.56	2472.31	62549.44
8	28.8	513.71	14794.85	2306.64	66422.59
Total		5402.31	81890.05	12765.65	317817.58

Table 4: Distributed lateral forces to each 8.5 m length walls

Floor	Number of walls in x direction	Total storey shear force [kN]	Lateral force applied to each wall [kN]	With additional torsional effect [kN]
8	30	2306.34	76.88	96.10
7	30	4778.65	159.30	199.12
6	30	6908.94	230.30	287.88
5	30	8697.21	289.90	362.38
4	30	10143.46	338.90	422.63
3	30	11247.69	374.90	468.63
2	30	12009.91	400.30	500.38
1	30	12765.65	425.50	531.88

Table 5: Type and number of metal connectors employed in the design

	Ground	1	2	3	4	5	6	7

Angle bracket	16 TTF200	15 TTF200	15 TTF200	13 TTF200	12 TTF200	9 TTF200	6 TTF200	3 TTF200
Hold-down	12 WHT620	12 WHT540	12 WHT540	12 WHT540	12 WHT440	12 WHT440	12 WHT440	12 WHT440

3. Numerical Modelling

In order to investigate the seismic behaviour of 8-storey CLT building, the Finite Element Framework OpenSees [18] was used. CLT panels were modelled as linear elastic shell elements with elastic-isotropic wood material property of quad elements. The *ndMaterial-ElasticIsotropic* material model of OpenSees [18] was employed for this. Shear and uplift connectors were modelled using non-linear link and zero-length elements using hysteretic material properties available in OpenSees [18]. CLT shear wall panels were divided into 3 pieces and structural screws were employed to assemble vertical connections. In addition, since quad elements are defined with only 2 degrees of freedom per node, they do not lend themselves for geometric transformations other than linear effectively preventing the consideration of second order actions and deformations. Therefore, in order to account for P-delta effect, an elastic leaning column with line elements of 3 degrees of freedom per node working in tandem with the original CLT wall was defined and integrated to it via multi-point constraints. Figure 3 illustrates the numerical model and the hysteretic material definition employed for the connectors. In this figure, *sp* and *ep* are stress and strain (or force and deformation) at first, second and third point of the envelope in positive direction, *sn* and *en* are stress and strain (or force and deformation) at first, second and third point of the envelope in negative direction.

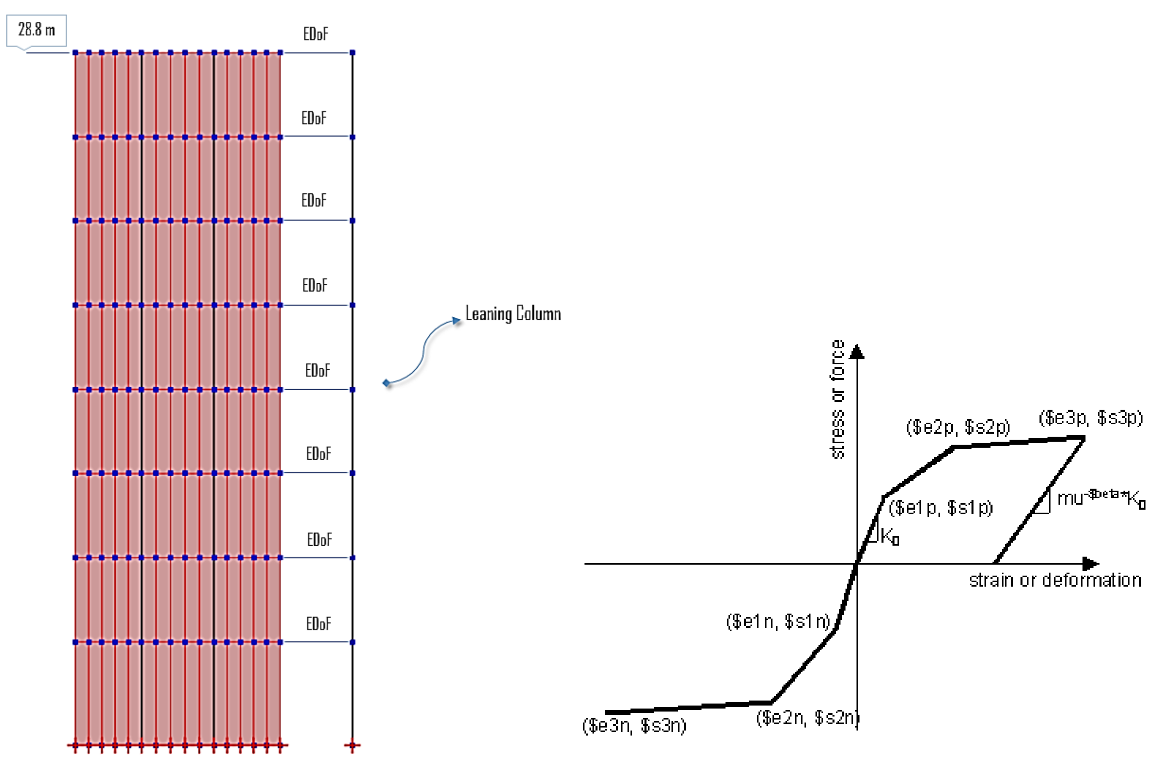
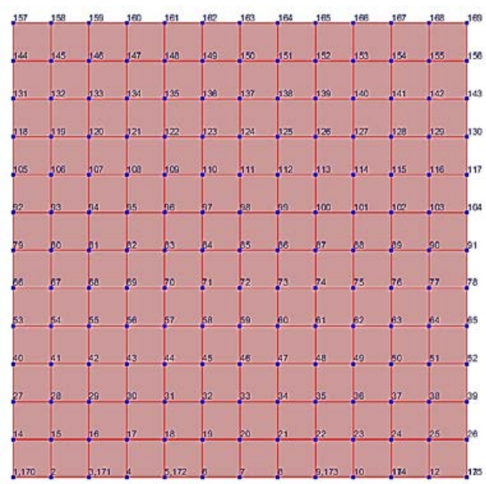


Figure 3: Numerical modelling of 8-Storey CLT building and hysteretic material definition in OpenSees [18]

4. Verification of the modelling approach

A single-panel and a double-panel CLT walls tested and modelled in Abaqus [21] by Gavric et al. [13] were modelled herein for verification purposes. The single-panel wall was connected to the foundation with four angle (shear) brackets and two hold-down (uplift) connectors at both ends. HTT22 hold-downs using Φ 4x60 mm nails in their vertical leg and connected to the foundation by means of M16 bolts were employed together with BMF 90x48x3x116 mm angle brackets (employing 11 Φ 4x60 mm nails) connected with M12 bolts to the foundation. The CLT panel was formed by five 17-mm thick layers. The wall panel dimensions were 2.95x2.95 m. The loading protocol for the reverse cyclic pushover analysis was the one prescribed by BS EN 12512 [22]. An additional 18.5 kN/m vertical load was applied on top of the panel. Figure 4 presents the numerical model developed in OpenSees [18] and loading protocol employed. Similarly, Figure 5 compares the numerical predictions and experimental results in terms of displacement versus shear force. A good level of approximation between the OpenSees [18] simulation and the Abaqus [21] and experimental results is evident from this figure.



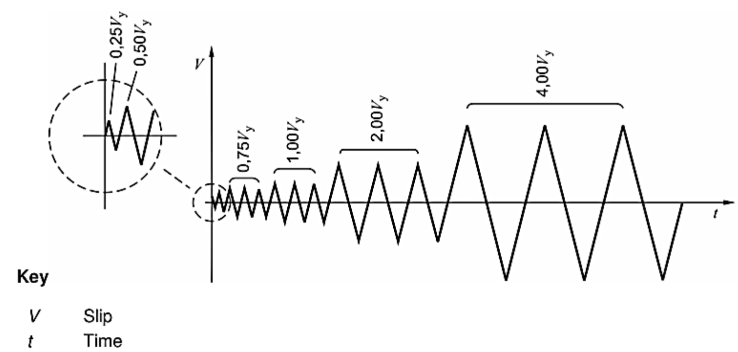


Figure 4: Numerical model of the CLT wall panel and Loading protocol in BS EN 12512 [22]

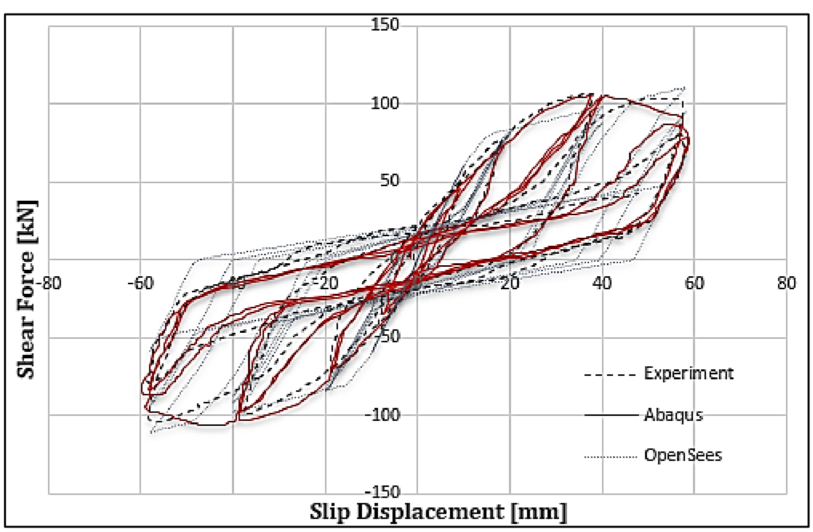


Figure 5: Comparison of numerical and experimental cyclic behaviour of a single CLT wall panel

5. Analyses and Results

5.1 Non-linear static (pushover) analysis

A nonlinear static analysis was performed on the numerical model of the 8-storey CLT building designed in Section 3 above. An inverted triangular load distribution was employed along the building height and increased monotonically until the onset of failure in the model. Figure 6 shows the results of the non-linear static analyses in terms of base shear against roof drift for the cases with and without leaning column. The influence of the P-Delta effects is clearly appreciated from Figure 6 where a decrease of around 5% in the base shear capacity of the structure can be observed when second order effects are adequately accounted for. The N2 [23, 24] method has been employed to assess non-linear static analysis results. Among several NLSA methods, the N2 method was accepted to present the best approximation for SDoF and MDoF systems [25].

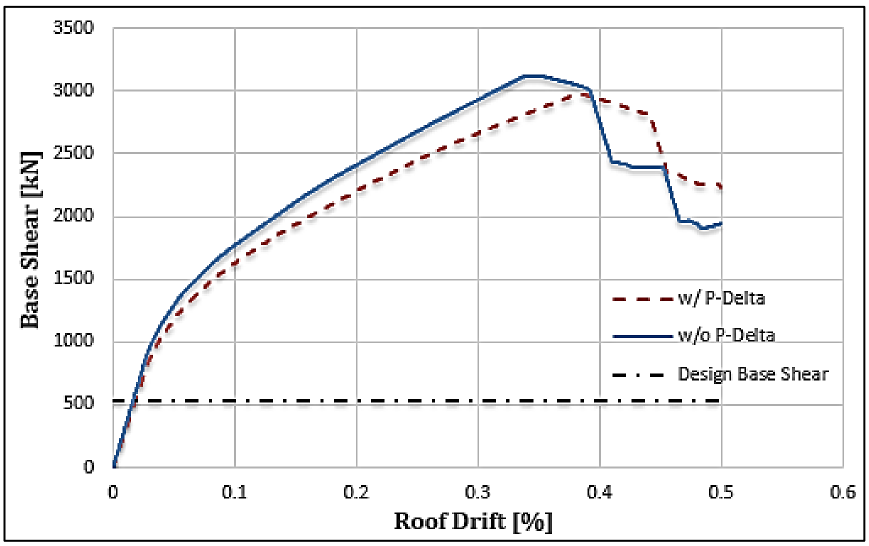


Figure 6: Non-linear static (pushover) analysis results of 8-storey CLT building

The procedure of the standard bi-linearization of the pushover curve recommended in the N2 method estimates the acquisition of the ultimate displacement is when the first structural component reaches the near collapse (NC) state. However, in the case of CLT, the near collapse state has been characterized as a global condition on the entire building with regard to Yasumura and Kawai [26] procedure. Thus, the NC state is considered to be obtained beyond the maximum base shear force of the global pushover curve. That is to say, the ultimate displacement is determined when the peak base shear force drops by 20% on the global pushover curve. To this end, the point of ultimate displacement corresponds to around 0.42% roof drift in the 8-storey CLT building.

5.2 Non-linear time history analysis

A total of 1270 ground motion records from 51 earthquakes with magnitudes Mw ranging from 5.65 to 7.90, and distances ranging from 0.44 to 413 km with the PGA of between 0.0033 and 1.43 g were employed. The acceleration records were obtained from the PEER-NGA database (http://peer.berkeley.edu/nga) and involve different site classes (according to the NEHRP classification). Figure 7 presents the distribution of Spectral Acceleration at the first period of the building, $S_a(T_1)$, against the ground-motion mean period, T_m. The mean period is used herein to characterise the frequency content of the ground-motion based on the studies of Málaga-Chuquitaype et al. [27, 28]. The mean period, T_m, was initially proposed by Rathje et al. [29] and is defined as the weighted mean of the periods of the Fourier Spectrum over a pre-defined frequency range such that:

$$T_m = \frac{\sum_i C_i^2/f_i}{\sum_i C_i^2}, \text{ for } 0.25 \text{ Hz} \leqslant f_i \leqslant 20 \text{ Hz, and with } \Delta f \leqslant 0.05 \text{ Hz}$$

where C_i is the Fourier Amplitude, corresponding to a frequency, f_i. It can be appreciated from Figure 7 that the selected ground-motion dataset presents a reasonably uniform distribution of $S_a(T_1)$ and T_m. More detailed information on the selected records can be found elsewhere [28].

Figures 8 to 10 below summarise the results of the non-linear response history analyses carried out. Figure 8 presents the relationship between spectral acceleration at the fundamental period of the building as a function of peak drift along the height of the building. The blue line on this figure represents the best power fit curve. The

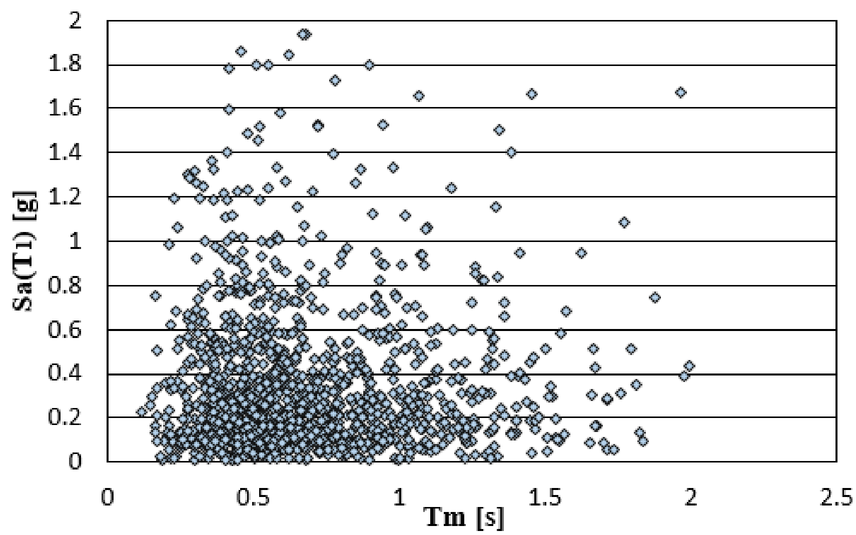


Figure 7: Distribution of spectral acceleration at the building's first period and ground-motion mean period for the dataset employed

relationship is non-linear and maximum peak drifts in the order of 9% were observed. Although significant scatter is evident from this figure, it can be seen that ground-motions with spectral accelerations as low as 0.05 g can lead to significant drifts in the order of 2% which represents an important inelastic deformation in the building. Similarly, larger spectral accelerations of 1.9 g can still lead to elastic response.

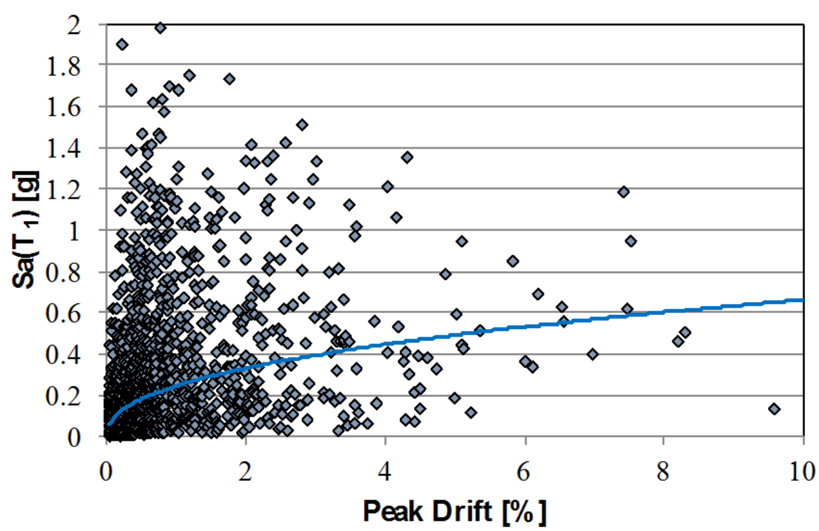


Figure 8: Relationship between spectral acceleration and peak drift along the height of the building

Figure 9 depicts the relationship between inelastic displacement ratios, Cr, and normalized periods, T/T_m. The inelastic displacement ratio Cr is defined as the ratio between the peak lateral inelastic displacements observed from the response history analysis divided by the peak lateral elastic displacement demand on an infinitely elastic SDOF system with the same mass and initial stiffness. Similarly, the normalized period is defined as the ratio between the period of the building associated with its first mode and the ground-motion mean period defined above. The curve for Cr presented in Figure 9 follows the general trends observed on other building typologies by other researchers [27] where inelastic displacement ratios increase as the structural period tends to zero although a higher proportion of Cr < 1 are evident for the building under study.

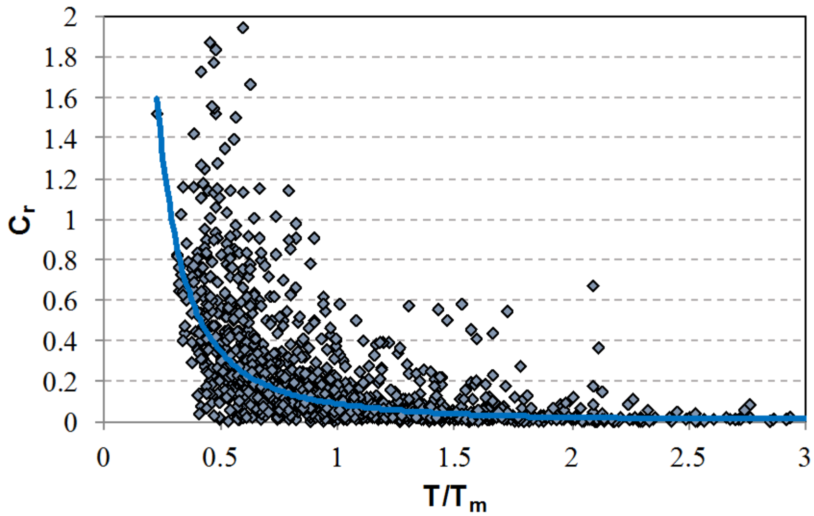


Figure 9: Inelastic displacement ratio against normalized period

Finally, Figure 10 presents the distribution of peak base shear against normalized period where the period normalization is the same as that described above. It can be observed from this figure that peak base shears of up to 2 times the design base shear can be developed, for T/T_m ratios between 0.25 and 0.75. Similarly, the linear trend line in Figure 10 indicates that the influence of the mean period on the peak base shear reduces along with the decrease of the mean period of ground motion records employed.

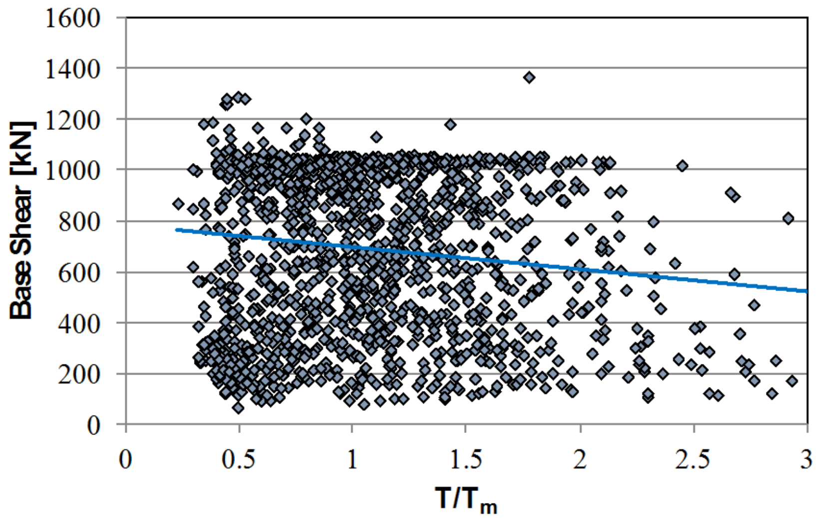


Figure 10: Peak base shear against normalized period

6. Conclusions

This paper has presented a detailed account of the seismic design of an 8-storey CLT building and its assessment through non-linear static and dynamic procedures. From these analyses, the following conclusions can be offered:

- Maximum values of peak drifts in the order of 9% were observed. Although significant scatter was evident, it could be appreciated that ground-motions with spectral accelerations as low as 0.05 g can lead to significant drifts in the order of 2% which represents an important inelastic deformation in the CLT building under consideration. Similarly, larger spectral accelerations of 1.9 g can still lead to elastic response.

- The relationship between inelastic displacement ratios, Cr, and period ratio (i.e. structure versus ground-motion) for the building under study follows the general trends observed for other building types in steel and concrete where inelastic displacement ratios increase as the structural period tends to zero. Nevertheless, a higher proportion of Cr < 1 are evident for the 8-storey CLT building under study.

- Peak base shears of up to 2 times the design base shear have been observed during the response history analyses, especially for T/T_m ratios between 0.25 and 0.75. Similarly, it can be seen that the influence of the mean period on the peak base shear reduces as the mean period of ground motion records decrease.

These findings call for further studies on the characterization of the response of high-rise CLT structures considering other ground-motion characterization parameters.

7. Acknowledgements

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