SECED 2015 Conference: Earthquake Risk and Engineering towards a Resilient World
9-10 July 2015, Cambridge UK

DYNAMIC RESPONSE OF TALL TIMBER BUILDINGS

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Abstract: The low carbon footprint and high structural efficiency of engineered wood materials make tall-timber buildings an attractive option for high-rise construction. However, due to the relatively low mass and stiffness characteristics of timber structures, some concerns have been raised regarding their dynamic response. This paper examines the dynamic behaviour of tall timber buildings under tornado and downburst wind loads. It summarizes the results of extensive response history analyses over a suite of FE structural models subjected to different wind actions and compares them with the ISO10137 comfort criteria. In general, large levels of floor accelerations are observed in particular for stiffer medium-rise structures with significant density of walls. It is shown that downburst loading governs the peak acceleration response of medium-rise buildings whilst tornado loading becomes more critical for taller buildings. The effectiveness of TMDs in reducing peak acceleration values is explored. This study emphasizes the need for further studies on the dynamic behaviour of tall timber buildings.

Introduction

The last 60 years have seen a significant increase in the number of high-rise buildings fostered by the sustained urban population growth around the globe (CTBUH, 2015). In this context, tall timber buildings have the potential to become the most efficient solution given the well-established environmental, social and economical advantages of wood and wood-based materials. The UK has been leading the tall timber trend with the most notable examples being the 9-storey Stadthaus building (Thompson, 2009) and the 8-storey Bridport House (EURBAN, 2015), recently superseded in height by the a 32-meter tall apartment building in Australia (Forte, 2014) which is the tallest timber building to date. However, taller timber buildings are in prospect with a 30-storey prototype design by the Canadian companies MGB Architecture and Equilibrium Consulting (Green, 2012); and a recently finished feasibility study by the American company Skidmore, Owings and Merrill for a 42-storey building (SOM, 2014), both of which employ Cross-Laminated-Timber (CLT) cores as main lateral resisting systems.

Chapman et al. (2012) performed an analysis of the dynamic response of a 30-level CLT building under wind actions following the recommendations of Eurocode (CEN, 2004). The proposed building had a circular CLT core with column rows radiating outwards. Peak drifts and accelerations of the structure were estimated based on equivalent static procedures. The study concluded that the 30-storey massive timber building was structurally feasible and that it satisfied all Ultimate Limit States (ULS) and Serviceability Limit State (SLS) performance criteria. In contrast, the investigation by Utne (2012) on a shorter 14-storey luxury residential building to be built in Norway concluded that the level of accelerations experienced by the building might be higher than the acceptability criteria outlined in the ISO10137 standard (ISO, 2012). Similarly, Reynolds et al. (2011) highlighted the possibility of unacceptable dynamic response of tall timber buildings due, primarily, to their light weight and stressed the importance of gaining a deeper understanding of the stiffness and damping characteristics exhibited by these structures.

This paper aims at assessing the vibration response of tall-timber buildings when subjected to loads arising from different wind velocity profiles. To this end, four recently proposed tall-

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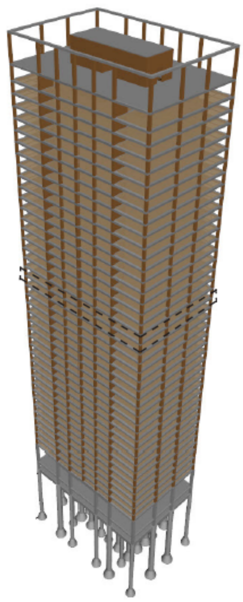
timber buildings are modelled by means of advanced FE procedures and their response under transient wind loads is analysed. The performance level of interest in this evaluation is related to vibration comfort and therefore, severe structural deterioration and fatigue are not considered. Two wind velocity profiles are taken into account, namely tornado and downburst for which maximum relative acceleration levels are calculated. This paper highlights the general difficulties found when trying to satisfy the ISO10137 (ISO, 2012) comfort criteria. It also shows that downburst loading governs the peak acceleration response of mid-rise buildings whilst tornado loading becomes more critical for taller buildings. The use of Tuned-Mass-Dampers (TMDs) to reduce peak acceleration values is explored. It is shown that a TMD with an active mass of 5% the total building mass is able to reduce peak accelerations in up to 50%.

Building Systems and Models

Over the last decade, several innovative systems have been developed to promote the use of timber in high-rise construction. Due to the relatively stiff response of Cross-Laminated-Timber (CLT) panels, most of the proposed schemes will involve CLT cores or walls as part of their lateral resisting system and displacement control strategies. Figure 1 presents a view of two of such building proposals: i) panelised construction (Figure 1a) suitable for medium-rise residential buildings that can accommodate a significant number of walls, and ii) hybrid moment-resisting-frame with CLT cores that favours a more open building layout. Given their relevance and importance, the dynamic behaviour of these two forms of construction will be studied herein.

a) 7-storey CLT building (Ceccotti et al., 2013)



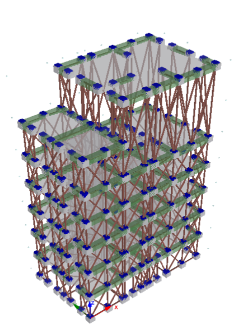
b) 30-storey timber building with CLT cores, glulam beam-columns and reinforced concrete joints (SOM, 2014)

Figure 1. Tall timber building systems

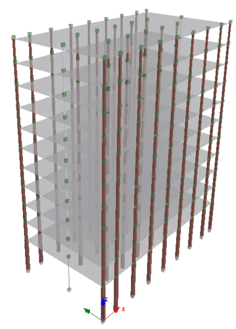
Full scale shaking table tests have been conducted on the 7-storey building of Figure 1a as part of the SOFIE project (Ceccotti et al., 2013). The blueprints of this building, as published by Dujic et al. (2010) and Ceccotti et. al. (2013), constitute the most detailed design drawings of a CLT building available in the literature and have been considered as a starting point for the development of Model B1 in this study. The SOFIE project also included investigations into the hysteretic behaviour of individual CLT panels, hold-downs and shear anchors which have informed the modelling of the connections as detailed below.

An alternative tall-timber system is that proposed by SOM (2014)which utilizes concrete joints in order to effectively transfer forces and moments between individual timber components. Additionally, the first floor is made of concrete for durability considerations whereas a CLT core is employed for lateral stability. A detailed set of structural drawings and connection details has been published for this building proposal (SOM, 2014) and form the basis for the modelling assumptions implemented as part of the present study.

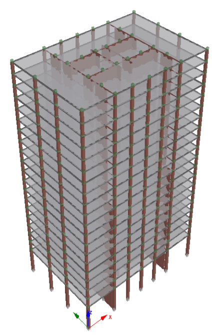
A total of 4 building models ranging from 7 to 30 storeys tall and including CLT panelised construction as well as framed systems were constructed. Figure 2 presents the Finite Element (FE) models developed and implemented in the advanced nonlinear software Seismostruct (Seismosoft, 2013). Due to the high relative stiffness of CLT panels in comparison with the typical steel connections employed, a rigid truss was employed to model each individual CLT panel for the 7 storey building (Ceccotti et al., 2006) as detailed in Figure 3. Model B1 (Figure 3a) is based on the research performed as part of the SOFIE project (Ceccotti et al., 2013). Contact elements were employed to simulate the rocking and sliding between adjacent panels and between the CLT panels and the flooring system. Distributed masses were attached to the linear flooring elements considering a 30% contribution of the superimposed loads. All nonlinearity was concentrated at the connections which were modelled as zero-length links with force-displacement characteristics replicating those observed during the experiments (Dujic et al., 2010).



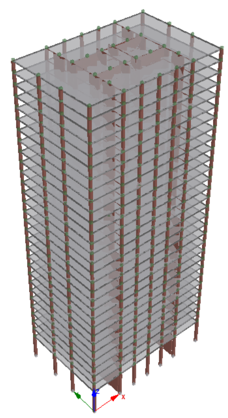
a) Building B1, 7 storeys



b) Building B2, 11 storeys



c) Building B3, 20 storeys



d) Building B4, 30 storeys

Figure 2. FE models of the buildings under consideration

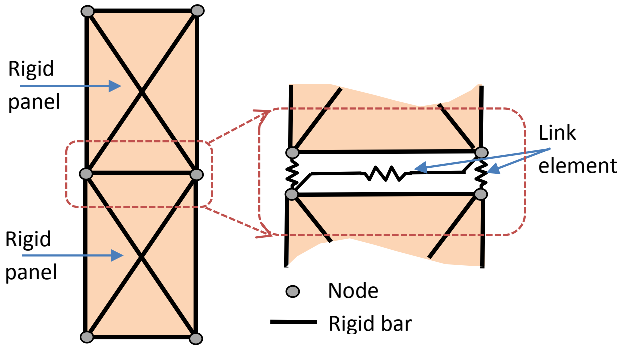


Figure 3. Modelling of CLT panels

Model B2 is based on a design carried out as part of a third-year group project by students of Imperial College London. This building relies on two CLT cores and employs Glulam columns and beams. The floor slabs are formed of 200 mm thick CLT panels with additional concrete topping. All connections are considered fully rigid. The building has 11 storeys and plan dimensions of 40 x 22 m.

Models B3 and B4 with 20 and 30 storeys, respectively, are roughly based on the layout proposed by SOM (2014). CLT floor slabs are considered whilst the columns are made of Glulam GL24. The column to column and wall to wall connections across floor levels are achieved by means of reinforced concrete spandrels and link beams where full rigidity and strength was assumed for modelling purposes. Table 1 presents the material characteristics assumed for all timber elements. A C40/C50 concrete material was assumed when concrete was present with an elastic modulus of 10^{11} kPa.

Table 1. Timber characteristics

	Young's modulus [kPa]	Weight [kN/m^3]
CLT	9.652×10^6	4.905
Glulam	9.308×10^6	4.4145

Rigid diaphragm constraints were imposed at the floor level in all cases by means of penalty functions as opposed to Lagrange multipliers due to their relative computational efficiency. An initial calibration of the models was performed in light of the measured vibration response when available. The periods corresponding to the first and fourth modes for all numerical models considered are summarized in Table 2. Analyses were conducted for viscous damping values between 7 and 23% of the critical damping corresponding to the hold downs and shear anchors used in the construction of CLT structures as suggested by Gavric et al. (2014), but only the results for 13% of the critical viscous damping are presented herein. A more detailed description of the modelling assumptions can be found in Abeysekera (2014).

Table 2. Building periods

Building	Mode 1	Mode 4
B1	0.68	0.16
B2	1.17	0.28
B3	1.24	0.38
B4	1.76	0.54

Wind loading and analysis
Figure 4 presents the two wind velocity profiles considered in the present study. The Vicroy model (Vicroy, 1992) was assumed for the downburst profile. A maximum height of 60 m was assumed for the calculations following Savory et al. (2001) while a peak velocity of 21.5 m/s was presumed. Similarly, the wind velocity profile suggested by the Eurocode (CEN, 2004) was assumed for tornado loading.

Each building was subject to the two wind profiles outlined above and presented in Figure 4. For this, randomly generated wind speeds were modified to fit the assumed profiles along its height. These velocities were then used to obtain histories of wind induced forces along a single face of each building. The sampling rates were verified to be within the limits imposed by the Nyquist frequency and the 10th modal frequency of each building. Rayleigh damping was assumed in all cases and the full history of displacements, relative velocities and floor accelerations was calculated by means Seismostruct (Seismosoft, 2013). Additional analyses were performed considering a TMD located at the top of each building. The active mass was assumed to be 5% of the total building mass in all cases and no design optimization was performed.

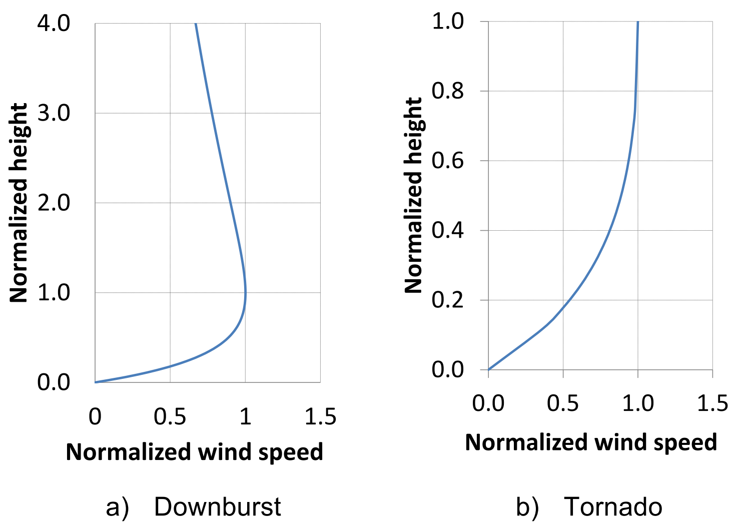


Figure 4. Normalized wind velocity profiles

Results
The response of the buildings was monitored at all floor levels in terms of displacements, relative velocities and relative acceleration histories. The peak acceleration was henceforth selected as the maximum value observed for the full duration of the analysis along the height of the building. It was found that for all models under both loading scenarios, the peak acceleration response took place at the roof level. This is in line with results of the modal analysis that indicated a first-mode dominated response in all cases. The response histories were then filtered by means of a band pass filter implemented in SeismoSignal in order to obtain peak acceleration levels for selected frequency ranges. Figure 5 and 6 summarize the results in terms of peak acceleration spectra for downburst and tornado loading, respectively. Also depicted in Figures 5 and 6 are the limits of peak acceleration suggested by ISO10137 for office and residential buildings.

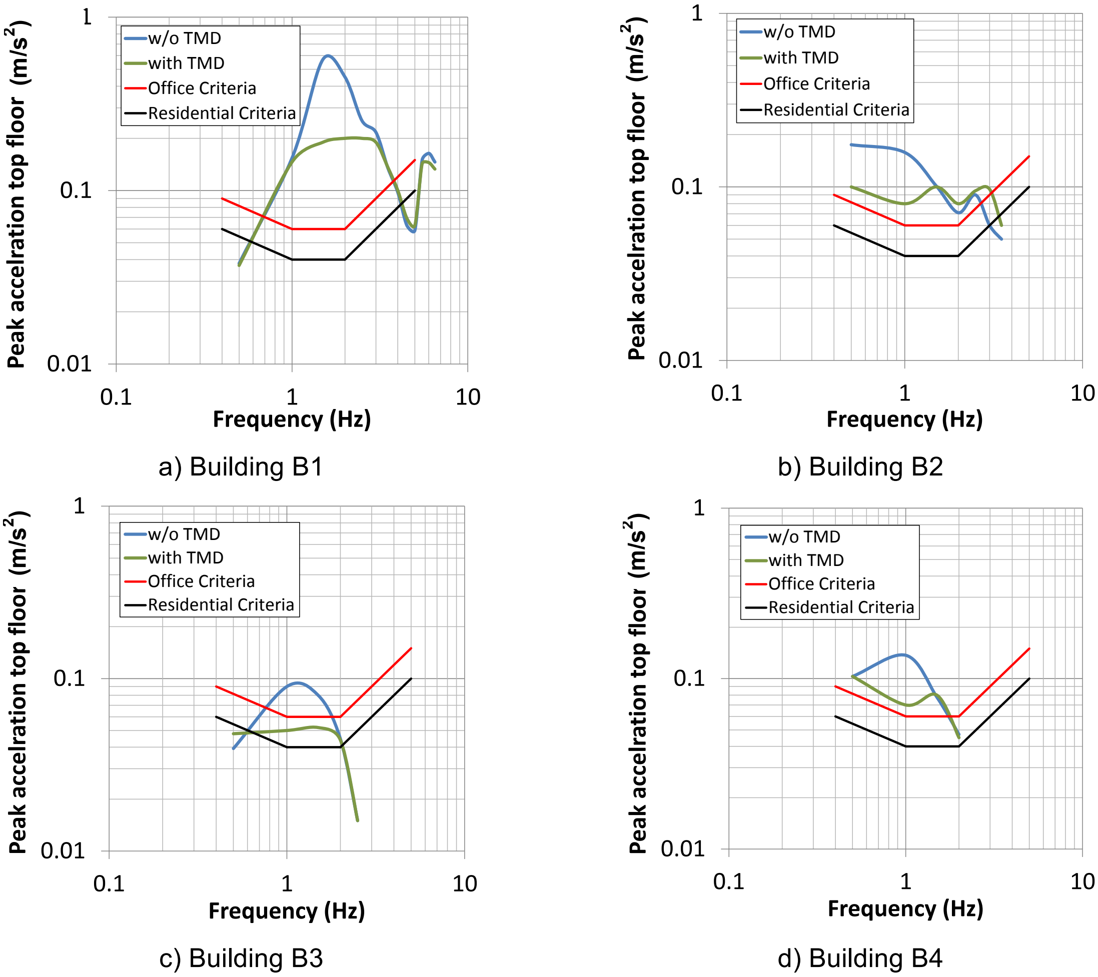
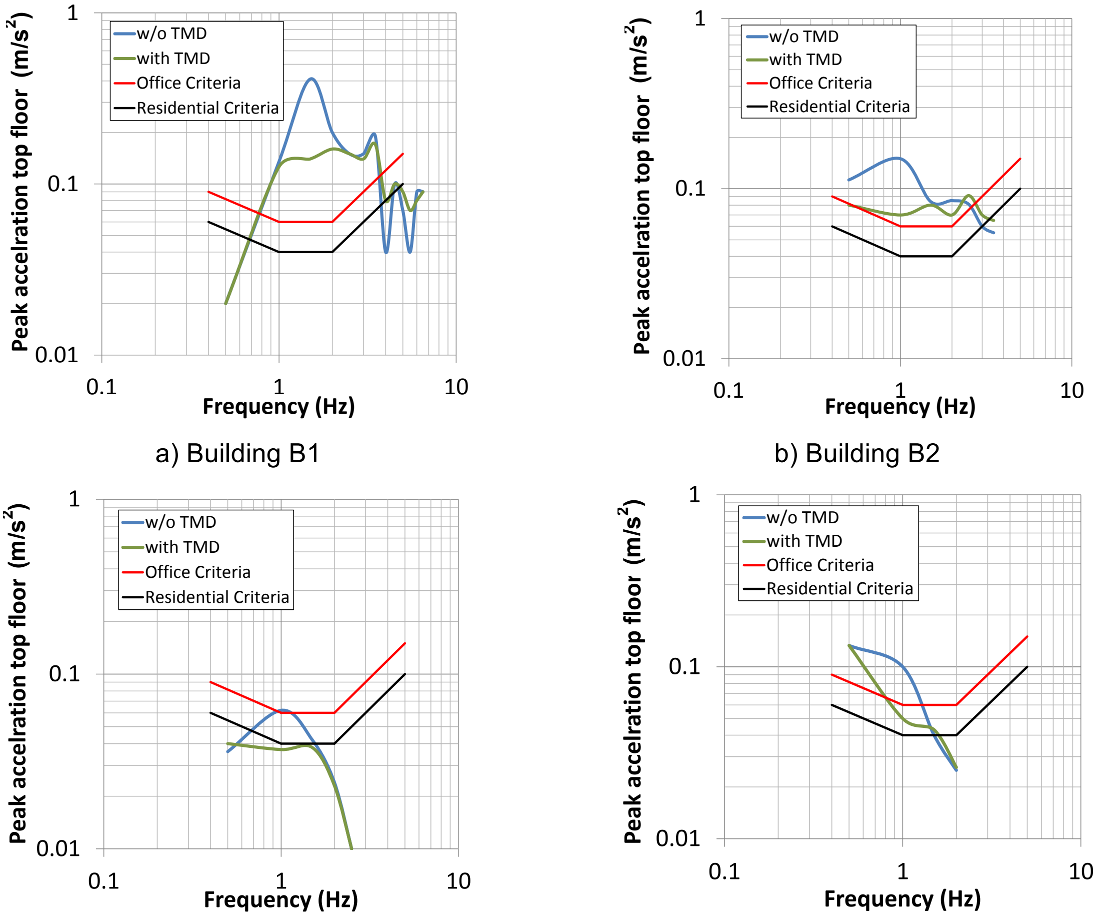


Figure 5. Peak floor accelerations for downburst action

It can be appreciated from Figures 5 and 6 that the models here studied do not meet the performance criteria outlined in ISO10137. In general, peak accelerations higher than the suggested ISO10137 thresholds are observed for all buildings around the frequency range associated with their first mode expect for building B4 under tornado loading. Only results for damping ratios of 13% are presented in Figures 5 and 6, nevertheless, it was observed that increasing the damping ratio in 5% can lead to reductions of around 30% in relative accelerations. The effects of damping seem to be more effective at low frequencies. It can also be seen from Figures 5 and 6 that the contribution of the first mode is greater than that of the second mode and that the ratio of their corresponding peak accelerations is in direct relationship with their effective modal masses.

Comparison between Figures 5 and 6 shows that for all models here considered downburst loading is more critical than tornado loading in generating peak acceleration demands. This is expected since downburst winds attain maximum speeds at lower altitudes than tornados. It can also be appreciated that shorter buildings (Models B1 and B2) have two distinct peaks reflecting the first and second mode contributions in the direction of loading. By contrast, taller buildings (models B3 and B4) present only one clearly defined peak which can be attributed to the nature of the load input. Besides, it can be observed that under downburst loading the peak response of Models B1 and B2 is increased to a greater degree than for models B3 and B4.



a) Building B1

b) Building B2

c) Building B3

d) Building B4

Figure 6. Peak floor accelerations for tornado action

Conclusions

This paper has examined the dynamic behaviour of four FE models of tall timber buildings under tornado and downburst wind loads. In general, large levels of floor accelerations were observed. The severity of the calculated peak accelerations was particularly large for the stiffer medium-rise CLT structure considered (Model B1). It was also shown that downburst loading is significantly more critical for the peak acceleration response of medium-rise buildings (Models B1 and B2) whilst tornado loading becomes comparably important for taller buildings (Models B3 and B4). Whilst the damping is uncertain and can vary widely for timber buildings an increase of 5% in the critical damping ration can reduce peak accelerations in up to 30%. However, this significant reduction is not enough to meet the international comfort criteria in most cases. The effectiveness of TMDs in reducing peak acceleration values is explored. This study emphasizes the need for further studies on the dynamic behaviour of tall timber buildings with attention to innovative dynamic response modification devices and details.

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