

# Numerical Analysis on Global Serviceability Behaviours of Tall Glulam Frame Buildings to the Eurocodes and UK National Annexes

Xuan Zhao<sup>1</sup>, Binsheng Zhang<sup>1\*</sup>, Tony Kilpatrick<sup>1</sup>, Iain Sanderson<sup>1</sup>, Dewen Liu<sup>2</sup>

1. Department of Civil Engineering and Environmental Management, School of Computing, Engineering and Built Environment, Glasgow Caledonian University, Glasgow G4 0BA, Scotland, UK

2. College of Civil Engineering, Southwest Forestry University, Kunming 650224, Yunnan Province, China  
E-mail: Ben.Zhang@gcu.ac.uk

Received: 31 January 2021; Accepted: 5 March 2021; Available online: 10 May 2021

**Abstract:** Glued-laminated timber (Glulam) is an innovative engineered timber product and has been widely used for constructing spatial grand timber structures and tall timber buildings due to its exceptional natural attraction, easy processing, decent fire resistance and outstanding structural performance. However, global serviceability performances of tall timber buildings constructed from Glulam products for beams, columns and bracings and CLT products for lift core and floors under wind load are not well known yet though they are crucial in structural design and global analysis. In this study, finite element software SAP2000 is used to numerically simulate the global static and dynamic serviceability behaviours of a 105 m high 30-storey tall Glulam building with CLT lift core and floors assumed in Glasgow, Scotland, UK. The maximum horizontal storey displacement due to wind is 58.5% of the design limit and the maximum global horizontal displacement is 49.7% of the limit set to the Eurocodes. The first three lowest vibrational frequencies, modes and shapes of the building are obtained, with the fundamental frequency being 33.3% smaller than the code recommended value due to its low mass and stiffness. The peak acceleration of the building due to wind is determined to the Eurocodes and ISO 10137. The results show that the global serviceability behaviours of the building satisfy the requirements of the Eurocodes and other design standards. Parametric studies on the peak accelerations of the tall Glulam building are also conducted by varying timber material properties and building masses. Increasing the timber grade for CLT members, the generalised building mass and the generalised building stiffness can all be adopted to lower the peak accelerations at the top level of the building so as to reduce the human perceptions to the wind induced vibrations with respect to the peak acceleration.

**Keywords:** Tall timber building; Glulam; Wind load; Serviceability; Storey displacement; Global horizontal displacement; Vibrational frequency and mode; Peak acceleration.

## 1. Introduction

Timber as a potential construction material has been used to build houses, schools, hotels, bridges, etc., everywhere in the world. At present, 90% of the 1,138,000 houses built annually are of timber structures in Northern America [1]. Timber can not only absorb CO<sub>2</sub> and but also help release more O<sub>2</sub>. Timber buildings can help reduce global CO<sub>2</sub> emissions [2,3]. Modern tall timber buildings have been precisely designed and promptly constructed all around the world. The third tallest timber building in the world is the 53 m high 18-storey UBC Brock Commons Student Residence in Vancouver, Canada, which could reduce CO<sub>2</sub> emission by 2432 tonnes [4], the second tallest timber building is the 84 m high 24-storey HoHo Tower in Vienna, Austria, which could reduce CO<sub>2</sub> emission by 2800 tonnes [5], and the tallest timber building at the moment is the 85.4 m 18-storey Mjøsa Tower in Brumunddal, Norway [6]. Most tall timber buildings use Cross laminated timber (CLT) products as the internal cores and floors and Glued laminated timber (Glulam) products as the frame-structural members.

Timber building structures have lower stiffness and mass density compared with the concrete and steel building structures, and their dynamic behaviours are very important for designers. For tall timber buildings, the design base shear forces due to wind loading are normally larger than those due to earthquake loading because of their high flexibility [7]. Abrahamsen et al. have experimentally explored the vibration behaviours of some real tall timber buildings in Europe and tried to predict the vibration responses of the tall timber buildings under wind loading through finite element modelling [8]. Aloisio et al. studied an eight-storey CLT building using a simplified analytical model and calibrated the experimental modal parameters using a proper objective function based on the storey masses of the building. The research output of the vibration frequencies of the first three modes showed that the damping factor of the building is close to that of the timber material itself. They also proved that the effect

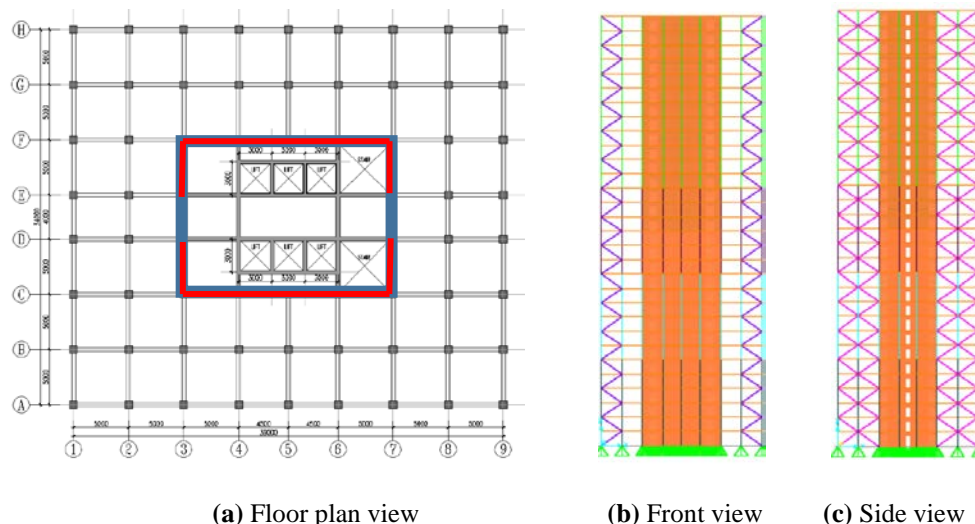
caused by connections could be negligible for low-amplitude dynamic responses [9]. In the analysis of the dynamic properties, Edskär and Lidelöw mentioned that the natural frequency is a very important parameter and the empirical formulae should be used with care for the analysis on timber structures [10].

In the research of tall timber buildings under wind load, dynamic behaviours related to acceleration levels at the top of the buildings are significantly crucial. Bjertnæs and Malo conducted a finite element analysis on the 14-storey timber building Treet in Norway and confirmed that the calculated maximum acceleration on the top floor of the building due to wind loading with a one-year return was slightly higher than the recommended value but was still considered to be acceptable. Research has been planned to install accelerometers to monitor wind velocities at different levels of the building [11]. Johansson et al. conducted a finite element analysis on a 22-storey timber building constructed from combined Glulam and CLT [12]. The results indicated that the calculated fundamental vibrational frequency of the model was very close to the estimated value using the method given in BS EN 1991-1-4 [13]. However, the calculated peak acceleration failed to satisfy the requirements specified in ISO 10137 [14].

Up to now, the research on wind-induced vibrations for tall timber buildings is still rarely reported. To understand the dynamic performances of tall timber buildings under lateral wind loading, this research aims to numerically investigate the global structural behaviours of a 105 m high 30-storey tall Glulam building under wind loading using commercial finite element analysis software SAP2000 [15]. The natural frequencies and modal shapes of the first three vibration modes and the lateral storey and global displacements of the building can be obtained from the finite element modelling. The detailed analysing procedure and the obtained results for the peak acceleration at the top level of the building are illustrated. Extensive investigations into the relationships of the peak accelerations of the tall Glulam buildings with timber grade, mass and stiffness are also conducted.

## 2. Basic plans of the tall Glulam frame building incorporated with CLT core and floors

The investigated tall Glulam frame building is a 30-storey office building, and for convenience it is assumed to be located in Glasgow, Scotland, UK. The finite element model for the building is established using SAP2000, and only linear geometrical analysis is considered in this research. The plan dimensions of the building are 39 m  $\times$  34 m. The storey height is 3.5 meters, giving the overall building height of 105 m. The Glulam used for this frame building is GL24c according to BS EN 14080 [16], while the CLT used for this building is C24 softwood according to BS EN 338 [17]. Figure 1 illustrates the floor plan view, front view and side view of this tall Glulam frame building. The building has the same floor plan for every storey except the dimensions of vertical Glulam structural members.



**Figure 1.** Different views of the 105 m high 30-storey tall Glulam frame building

In Figure 1, the blue rectangular line indicates Wall 1 which is the 360 mm thick 9-layer uniform CLT inner core. The red line on the blue line indicates that there are no huge openings on the core wall. The whole building is designed as a structure with an inner CLT core and Glulam frames. Inside the core, the grey lines indicate the shear walls for the building, with Wall 2 as those 360 mm thick 9-layer walls between Storeys 1 and 15 and Wall 3 for those 280 mm thick 7-layer walls above. All floor slabs for this building are constructed from 245 mm thick 7-player CLT panels. The Glulam frame structure is constructed around the core with Glulam columns with the

sizes varying from Storeys 1 to 30. The dimensions of the Glulam members are listed in Table 1. The dimensions of the Glulam columns are 800 mm × 800 mm, 700 mm × 700 mm, 600 mm × 600 mm and 500 mm × 500 mm, respectively. The dimensions of the Glulam beams are 350 mm × 500 mm, and all the beams are assumed to be pin-connected to the columns. The dimensions of the bracing members in both y- and x-directions are 400 mm × 400 mm, and they are only located along lines 1, 9, A and H, and are all pinned to the column-beam joints, as shown in Figure 1. To reduce the large data caused by small element sizes and multi-layered materials in the finite element analysis modelling, the estimation method is to be used to simplify and convert multi-layer elements into single layer thin orthotropic shell elements of 500 mm × 500 mm [18]. Single CLT members can be verified to the Canadian CLT Handbook [19] and Wallner-Novak's book [20], and other timber members and connections can be verified to BS EN 1995-1-1 [21] and the UK National Annex (NA) [22] to BS EN 1995-1-1.

**Table 1.** Sizes of Glulam members for the tall Glulam frame building

Glulam members for the building	Size (mm)
Columns 1 between Storeys 1 and 6	800 × 800
Columns 2 between Storeys 7 and 12	700 × 700
Columns 3 between Storeys 13 and 21	600 × 600
Columns 4 between Storeys 22 and 30	500 × 500
Beams	350 × 500
Bracings	400 × 400

The effects of actual connections between structural timber members are not considered at this stage due to the limitation of the finite element analysis software and all CLT members are assumed to be rigidly connected to Glulam beams, columns and bracings with the rigid boundary conditions. In this building, the infill panels are considered as the linear loading on the simply supported beams, but the contributions of infill panels to stiffness are not considered when numerically simulating this Glulam frame building.

All the structural CLT panels are defined as orthotropic thin-shell elements based on the estimate method. The in-plane stiffnesses are applied to all the CLT wall members with the out-of-plane stiffnesses applied to all the CLT slab members. The calculated equivalent stiffness parameters of the CLT elements used in this research are listed in Table 2. Here,  $E_{rx}$  and  $E_{ry}$  are the equivalent bending elastic moduli about x- and y-axes for out-of-plane case,  $E_x$  and  $E_y$  are the equivalent bending elastic moduli about x- and y-axes for in-plane case,  $G_{zx}$  is the equivalent shear modulus in the zx-plane,  $G_{yz}$  is the equivalent shear modulus in the yz-plane, and  $G_{xy}$  is the equivalent shear modulus in the xy-plane. For Glulam elements, the bending elastic modulus parallel to the grain,  $E_0 = 11000$  N/mm<sup>2</sup>, the bending elastic modulus perpendicular to the grain,  $E_{90} = 300$  N/mm<sup>2</sup>, the shear modulus parallel to the grain is  $G_0 = 650$  N/mm<sup>2</sup>, and the shear modulus perpendicular to the grain is  $G_{90} = 65$  N/mm<sup>2</sup>. Rigid diaphragms are assumed for all CLT floor slabs. The wind load is defined as exposure from extents of rigid diaphragms in a wind direction of 270°. The wind velocity is 26.21 m/s, the terrain category is set as II, and the orography factor, turbulence factor and structural factor are all set as 1.0. The dead load of 0.92 kN/m<sup>2</sup> excluding the self-weight of the floor is applied on the floor, together with the imposed load of 3.3 kN/m<sup>2</sup>. The dead load from the plasterboards and the assembling and decorating parts on the Glulam beams is estimated as 5.38 kN/m. As shown in Figure 1, there are some areas used as stairs. To assemble these areas to the model, the thickness of the areas is adopted as 0 mm, with the dead load estimated as 4.0 kN/m<sup>2</sup> and the imposed load as 3.0 kN/m<sup>2</sup>. The applied actions on the building are chosen from BS EN 1991-1-1 [23] and the corresponding UK NA [24]. The load combinations are in accordance to BS EN 1990 [25] and the corresponding UK NA [26].

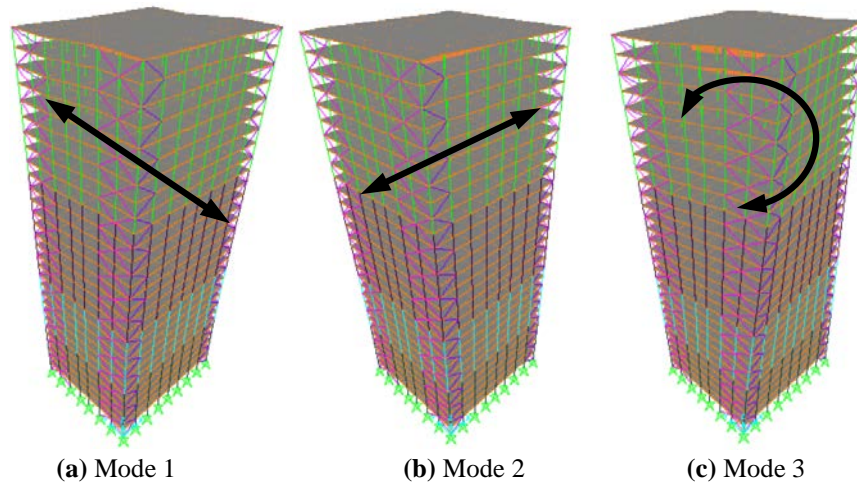
**Table 2.** Equivalent stiffness parameters of the CLT elements in the tall Glulam frame building

Stiffness parameters	Walls 1 and 2	Wall 3	Floor slab
$t$	360.0 mm	280.0 mm	245.0 mm
$E_{rx}$	7441 N/mm <sup>2</sup>	7933 N/mm <sup>2</sup>	7929 N/mm <sup>2</sup>
$E_{ry}$	3927 N/mm <sup>2</sup>	3438 N/mm <sup>2</sup>	3436 N/mm <sup>2</sup>
$E_x$	5094 N/mm <sup>2</sup>	4926 N/mm <sup>2</sup>	4926 N/mm <sup>2</sup>
$E_y$	6276 N/mm <sup>2</sup>	6444 N/mm <sup>2</sup>	6444 N/mm <sup>2</sup>
$G_{xy}$	392.22 N/mm <sup>2</sup>	402.77 N/mm <sup>2</sup>	402.77 N/mm <sup>2</sup>
$G_{yz}$	111.53 N/mm <sup>2</sup>	107.54 N/mm <sup>2</sup>	107.55 N/mm <sup>2</sup>
$G_{zx}$	111.53 N/mm <sup>2</sup>	107.54 N/mm <sup>2</sup>	107.55 N/mm <sup>2</sup>

### 3. Global performances of the tall Glulam frame building

#### 3.1 Vibrational frequencies, modes and shapes

After running the finite element building model for the tall Glulam frame building, the modal frequencies and shapes for the first three lowest modes are obtained and illustrated in Figure 2. The first vibrational mode occurs along the short building side direction with the modal frequency of 0.292 Hz. The second vibrational mode occurs along the long building side direction with the modal frequency of 0.322 Hz. The third vibrational mode is the rotational mode about the vertical axis with the modal frequency of 0.386 Hz. The first three vibrational modes of the building are the primarily considered ones for structural engineers to check when designing tall reinforced concrete or steel buildings. In this study, the same three lowest vibrational modes including modal frequencies and shapes are considered for analysing the tall Glulam frame building.



**Figure 2.** The first three lowest vibrational modes for the tall Glulam frame building

From BS EN 1991-1-4, the fundamental flexural frequency of a multi-storey building,  $f_1$ , for a height greater than 50 metres can be estimated as

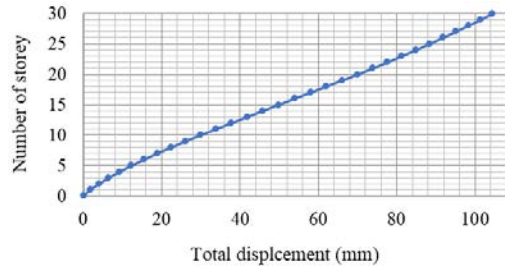
$$f_1 = \frac{46}{h} \quad (1)$$

This empirical formula is obtained from analyses on huge experimental results on tall concrete and steel building structures. Based on this, the fundamental frequency of the currently modelled tall Glulam frame building should be estimated as  $f_1 = 46/105 = 0.438$  Hz. In this study, the lowest modal frequency of the building is numerically obtained as 0.292 Hz, which is 33.3% smaller than the code recommended value. This means that the building is much less stiff than expected. A number of factors may contribute to the difference in the fundamental frequency but are mainly associated with the stiffness and mass of the building. The CLT core in this building contains many pre-assumed door openings on the CLT walls and lift core. The main part in this building is a frame structure without CLT walls. Also, this building is assumed to have rigid connections between the CLT members by disregarding the flexibility effects of the connections. Using the formula from Reynolds et al. [27], the fundamental frequency is estimated as  $f_1 = 55/105 = 0.524$  Hz, and the numerically estimated value in this study is 44.3% smaller than the recommended value by Reynolds et al. At this moment, however, there is no restriction on the fundamental frequency in the current structural Eurocodes, indicating that the fundamental frequency of this tall timber building is still acceptable.

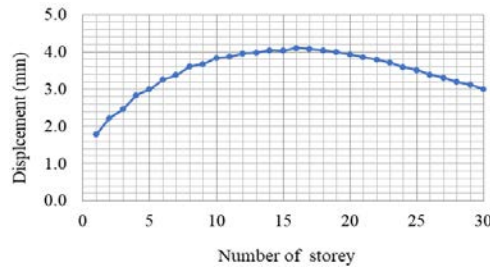
#### 3.2 Static storey and global horizontal displacements

Serviceability criteria for buildings and other civil engineering work in terms of wind-induced horizontal deflections or displacements are not uniquely defined among the European countries because they are partially subjective. The Eurocodes do not provide numerical design limits for wind-induced horizontal deflections but only specify that these should be agreed with clients for individual projects and may be defined in the National Annexes. Technically, the deflection limits should be defined independently of construction materials, but it has not been possible to harmonise among all European countries on all construction materials yet. It is generally accepted that a limit of  $H/500$  is appropriate for the wind-induced horizontal deflections of each storey in a multi-storey building or the structure as a whole for a multi-storey building where  $H$  is the storey height or the total building height [28]. Here, characteristic combinations for all applied loads on buildings are recommended.

The loading combination is based on BS EN 1990 and the corresponding UK NA. The horizontal displacements at various building heights of the finite element building model for this building caused by wind loading are illustrated in Figure 3. The maximum horizontal displacement of the building occurs at the top level of the building with the value of 104.36 mm. It is generally accepted that the limit for the maximum horizontal displacement of the building is  $H/500$  or  $105,000/500 = 210$  mm for this building. The numerically determined displacement for the building reaches only 49.7% of the limit of the Eurocodes. Figure 4 illustrates the horizontal storey drifts of the building. The maximum storey drift occurs for Storey 16 at 4.096 mm which is 58.5% of the limit.



**Figure 3.** Horizontal displacements at various storey heights of the tall Glulam frame building due to wind load



**Figure 4.** Horizontal storey drifts of the tall Glulam frame building due to wind load

## 4. Peak acceleration at the top level of the tall Glulam frame building due to wind load

### 4.1 Manual analysis on the peak acceleration at the top level of the building

BS EN 1991-1-4, the corresponding UK NA and ISO 10137 are the main design standards used in this research. The horizontal peak acceleration of the building structure,  $A(z)$ , is calculated as

$$A(z) = k_p \sigma_{a,x}(z) \quad (2)$$

where  $k_p$  is a peak factor. The term  $\sigma_{a,x}$  is the standard deviation of the characteristic along-wind acceleration of the structural point at height  $z$  and is obtained using Eq.(B.10) of BS EN 1991-1-4 as

$$\sigma_{a,x}(z) = \frac{c_f \rho b I_v(z_s) v_m^2(z_s)}{m_{1,x}} R K_x \Phi_{1,x}(z) \quad (3)$$

where,

- $c_f$  is the force coefficient,
- $\rho$  is the air density,
- $b$  is the width of the structure,
- $I_v(z_s)$  is the turbulence intensity at the height  $z = z_s$  above ground,
- $v_m(z_s)$  is the mean wind velocity for  $z = z_s$ ,
- $z_s$  is the reference height,
- $R$  is the square root of resonant response,
- $K_x$  is the non-dimensional coefficient,
- $\Phi_{1,x}(z)$  is the fundamental along-wind modal shape,
- $m_{1,x}$  is the along-wind fundamental equivalent mass.

The following details the manual calculations for determining the peak acceleration at the top level of the investigated tall Glulam frame building in accordance to BS EN 1991-1-4, the corresponding UK NA and ISO 10137.

The height of the building is 105 m, with the building width as 39 m and the building depth as 34 m. The force coefficient  $c_f$  is equal to the net pressure coefficient in the UK NA, and its value is 1.241. The air density in the UK is equal to 1.226 kg/m<sup>3</sup>. The turbulence intensity at the height  $z = z_s$  above ground is equal to 0.134. Thus, the mean wind velocity  $v_m(z)$  at  $z = z_s$  could be calculated from Eq.(4.3) of BS EN 1991-1-4 as

$$v_m(z) = c_r(z) c_o(z) v_b \quad (4)$$

According to Figure NA.3 in the UK NA to BS EN 1991-1-4 [29], the roughness factor  $c_r(z) = 1.37$  for  $h_{dis} = 0$  and a distance of 40 km from the site to the sea. In Clause NA.2.9, the orography factor  $c_o(z)$  is recommended as 1.0. Based on BS EN 1991-1-4 and the UK NA, the basic wind velocity  $v_b$  is determined as 26.21 m/s. Hence, the mean wind velocity  $v_m(z) = 1.37 \times 1.0 \times 26.21 = 35.91$  m/s.

Based on Clause 4.2(2) Note 4 in BS EN 1991-1-4, the 10-minute mean wind velocity having the probability  $p$  for an annual exceedance is determined by multiplying the basic wind velocity  $v_b$  by the probability factor  $c_{prob}$  which is determined from

$$c_{prob} = \left( \frac{1-K \ln(-\ln(1-p))}{1-K \ln(-\ln(0.98))} \right)^n = \left( \frac{1-0.2 \times \ln(-\ln(1-0.6321))}{1-0.2 \times \ln(-\ln(0.98))} \right)^{0.5} = 0.749 \quad (5)$$

where the probability  $p$  for an annual exceedance is reasonably assumed as  $1 - 1/e = 0.6321$  [30], the shape parameter  $K$  depending on the coefficient of variation of the extreme-value distribution is recommended as 0.2, and the exponent  $n$  is recommended as 0.5. Thus, the mean wind velocity for  $z = z_s$  according to a one-year return period is equal to  $v_m(z) = 35.91 \times 0.749 = 26.90$  m/s. The resonance response factor  $R$  can be obtained from Eq.(B.6) in EN 1991-1-4 as

$$R^2 = \frac{\pi^2}{2 \delta} S_L(z_s, n_{1,x}) R_h(\eta_h) R_b(\eta_b) \quad (6)$$

As shown in Clause F.5 of BS EN 1991-1-4, the total logarithmic decrement of damping,  $\delta$ , for the fundamental bending mode may be estimated from

$$\delta = \delta_s + \delta_a + \delta_d \quad (7)$$

where  $\delta_s$  is the logarithmic decrement of structural damping,  $\delta_a$  is the logarithmic decrement of aerodynamic damping for the fundamental mode, and  $\delta_d$  is the logarithmic decrement of damping due to special devices (tuned mass dampers, sloshing tanks, etc.). In Table F.2 of the code, for timber bridge,  $\delta_s$  could be chosen from 0.06 to 0.12. In this research, assume  $\delta_s = 0.06$  for timber buildings, and  $\delta_a$  can be obtained from Eq.(F.18) of the code as

$$\delta_a = \frac{c_f \rho b v_m(z_s)}{2 n_1 m_e} = \frac{1.241 \times 1.226 \times 39 \times 26.90}{2 \times 0.292 \times 216316.1} = 0.013 \quad (8)$$

Based on Clause F.4(2) in BS EN 1991-1-4 for cantilevered structures, the varying mass distribution  $m_e$  may be approximated by the average value of  $m$  over the upper third of the structure. For simplicity of the analysis, the average mass of the whole building is used for a varying mass distribution  $m_e$  as 216316.1 kg/m.

Hence,  $\delta = 0.06 + 0.013 + 0 = 0.073$ , where  $\delta_d$  is assumed to be 0.

The non-dimensional power spectral density function  $S_L(z, n)$  is given by Eq.(B.2) in BS EN 1991-1-4 as

$$S_L(z, n) = \frac{n S_v(z, n)}{\sigma_v^2} = \frac{6.8 \times f_L(z, n)}{(1 + 10.2 \times f_L(z, n))^{5/3}} \quad (9)$$

Here, the non-dimensional frequency  $f_L(z, n)$  is determined by using the frequency  $n = n_{1,x}$  based on Eq.(B.2) in BS EN 1991-1-4 as

$$f_L(z, n) = \frac{n L(z)}{v_m(z)} \quad (10)$$

where the frequency  $n = n_{1,x} = 0.292$  Hz. Also, based on Eq.(B.1) in BS EN 1991-1-4, the turbulent length scale  $L(z_s)$  can be obtained from

$$L(z_s) = \begin{cases} L_t \left(\frac{z}{z_t}\right)^\alpha & \text{for } z \geq z_{\min} \\ L(z_{\min}) & \text{for } z < z_{\min} \end{cases} \quad (11)$$

with

$$\alpha = 0.67 + 0.05 \ln(z_0) \quad (12)$$

In Table 4.1 of BS EN 1991-1-4, the roughness length  $z_0 = 0.05$  m and the minimum height  $z_{\min} = 2$  m, giving  $\alpha = 0.67 + 0.05 \times \ln(0.05) = 0.52$ . For  $z > z_{\min}$ , the reference height is  $z_t = 200$  m, and the reference length scale  $L_t = 300$  m. Thus, the turbulent length scale  $L(z_s)$  can be obtained as

$$L(z_s) = 300 \times \left(\frac{63}{200}\right)^{0.52} = 164.49 \text{ m}$$

The non-dimensional frequency  $f_L(z, n)$  is determined as

$$f_L(z, n) = \frac{n L(z)}{v_m(z)} = \frac{0.292 \times 164.49}{26.90} = 1.786$$

The non-dimensional power spectral density function  $S_L(z, n)$  is determined as

$$S_L(z, n) = \frac{6.8 \times 1.786}{(1 + 10.2 \times 1.786)^{5/3}} = 0.088$$

The aerodynamic admittance function about height,  $R_h$ , is given in Eq.(B.7) of BS EN 1991-1-4 as

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2} (1 - e^{-2\eta_h}) \quad (13)$$

with

$$\eta_h = \frac{4.6h}{L(z_s)} f_L(z_s, n_{1,x}) \quad (14)$$

Thus, the aerodynamic admittance function about height,  $R_h$ , can be obtained as

$$R_h = \frac{1}{5.244} - \frac{1}{2 \times 5.244^2} (1 - e^{-2 \times 5.244}) = 0.173 \text{ with } \eta_h = \frac{4.6 \times 105}{164.49} \times 1.786 = 5.244.$$

The aerodynamic admittance function about width,  $R_b$ , is given in Eq.(B.8) of BS EN 1991-1-4 as

$$R_b = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2} (1 - e^{-2\eta_b}) \quad (15)$$

with

$$\eta_b = \frac{4.6b}{L(z_s)} f_L(z_s, n_{1,x}) \quad (16)$$

Thus, the aerodynamic admittance function about width,  $R_b$ , can be obtained as

$$R_b = \frac{1}{1.948} - \frac{1}{2 \times 1.948^2} (1 - e^{-2 \times 1.948}) = 0.384$$

with

$$\eta_b = \frac{4.6 \times 39}{164.49} \times 1.786 = 1.948.$$

The resonance response factor  $R$  is determined as

$$R^2 = \frac{\pi^2}{2 \times 0.073} \times 0.088 \times 0.173 \times 0.384 = 0.395$$

According to Eq.(F.13) in BS EN 1991-1-4, the fundamental along-wind modal shape  $\Phi_1(z)$  is given as

$$\Phi_1(z) = \left(\frac{z}{h}\right)^\zeta \tag{17}$$

As shown in Clause F.3 of BS EN 1991-1-4, the parameter  $\zeta = 1.0$  for buildings with a central core plus peripheral columns or larger columns plus shear bracings, and the building height ratio  $z_s/z_0 = 63/0.05 = 1260$ . In Figure B.4 of the code, the non-dimensional coefficient  $K_x = 1.5$ . Thus, the fundamental along-wind modal shape  $\Phi_{1,x}(z) = 1$ , because  $z = h = 105$  m.

The standard deviation of the characteristic along-wind acceleration of the structural point at height  $z$ ,  $\sigma_a$ , can then be obtained as

$$\sigma_a(z) = \frac{1.241 \times 1.226 \times 39 \times 0.134 \times 26.9^2}{216316.1} \times \sqrt{0.395} \times 1.5 \times 1 = 0.0251.$$

In ISO 10137, the horizontal peak acceleration of the structure at height  $z$ ,  $A(z)$ , is given as  $A(z) = k_p \sigma_a(z)$ . Here, the peak factor  $k_p$  is calculated based on Eq.(B.4) in BS EN 1991-1-4 as

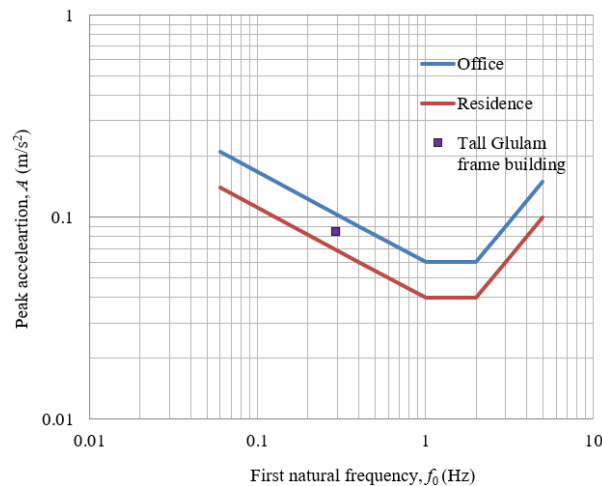
$$k_p = \max \left\{ \frac{\sqrt{2 \cdot \ln(v \cdot T)}}{3} + \frac{0.6}{\sqrt{2 \cdot \ln(v \cdot T)}} \right\} = \max \left\{ \frac{\sqrt{2 \cdot \ln(0.292 \times 600)}}{3} + \frac{0.6}{\sqrt{2 \cdot \ln(0.292 \times 600)}} \right\} = 3.401 \tag{18}$$

where the natural frequency is assumed to be the up-crossing frequency, i.e.  $v = n_{1,x} = 0.292$  Hz.

Therefore, the horizontal peak acceleration of the structure at height  $z$ ,  $A(z)$ , can be finally determined as

$$A(z) = k_p \sigma_a(z) = 3.401 \times 0.0251 = 0.085 \text{ m/s}^2.$$

According to ISO 10137, the calculated horizontal peak acceleration is illustrated in Figure 5, together with the design limits for both residential and office buildings. In the figure, the point for the peak acceleration is below the limiting line for office buildings, i.e. the blue line. Hence, the obtained acceleration at the top level of the building satisfies the requirement of ISO 10137.



**Figure 5.** The calculated wind-induced peak acceleration with the evaluation curves for residential and office buildings along a horizontal (x or y) direction for one-year return period

#### 4.2 Parametric studies on the peak accelerations of the tall Glulam frame building

The parameter studies on the peak accelerations of the tall Glulam frame building are divided into three parts. The first part explores the effect of the timber material grade, the second part explores the effect of the building mass, and the third and final part explores the effect of the building stiffness.



#### 4.2.1 The effect of the timber material grade on the peak acceleration of the tall Glulam frame building

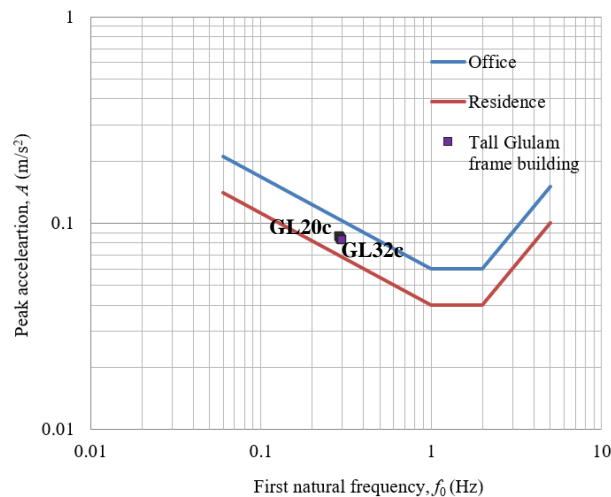
According to BS EN 338 and BS EN 14080, the grades of the timber materials for CLT and Glulam are not the same. For assessing the effect of the timber material grade on the peak accelerations of the investigated building, the analysing procedure should be divided into two parts.

The first part is that the new building models for this tall timber building are created by changing the timber material grade of Glulam members only. Seven different timber material grades are adopted, including GL20c, GL22c, GL24c, GL26c, GL28c, GL30c and GL32c, respectively. Model A1 uses GL20c timber for Glulam members, Model A2 uses GL22c timber, Model A uses GL24c timber, Model A3 uses GL26c timber, Model A4 uses GL28c timber, Model A5 uses GL30c timber, and Model A6 uses GL32c timber. After running different numerical models using SAP2000, the fundamental modal frequencies and peak accelerations at the top level of the building by varying only the timber material grade for the Glulam members from GL20c to GL32c can be determined and the obtained results are listed in Table 3.

Figure 6 illustrates the obtained peak accelerations at the top of the building versus the fundamental frequency by varying only the timber material grade of Glulam members from GL20c to GL32c, together with the evaluation curves for residential and office buildings. It can be seen that with the increasing timber grade for the Glulam materials only, the fundamental vibrational frequency of the tall Glulam frame building increases and the acceleration at the top level of the building decreases from  $0.086 \text{ m/s}^2$  to  $0.083 \text{ m/s}^2$ , down by 3.5%. Because the adopted timber grades are limited, the differences between the obtained results are very small. This indicates that changing timber materials grade of the Glulam members only is not a practical option to largely enhance the design of tall timber buildings.

**Table 3.** Parametric study on the peak acceleration by varying the timber material grade of Glulam members

Model	Grade	Frequency	Peak acceleration
A1	GL20c	0.290 Hz	$0.086 \text{ m/s}^2$
A2	GL22c	0.290 Hz	$0.086 \text{ m/s}^2$
A	GL24c	0.292 Hz	$0.085 \text{ m/s}^2$
A3	GL26c	0.295 Hz	$0.084 \text{ m/s}^2$
A4	GL28c	0.296 Hz	$0.084 \text{ m/s}^2$
A5	GL30c	0.297 Hz	$0.084 \text{ m/s}^2$
A6	GL32c	0.299 Hz	$0.083 \text{ m/s}^2$



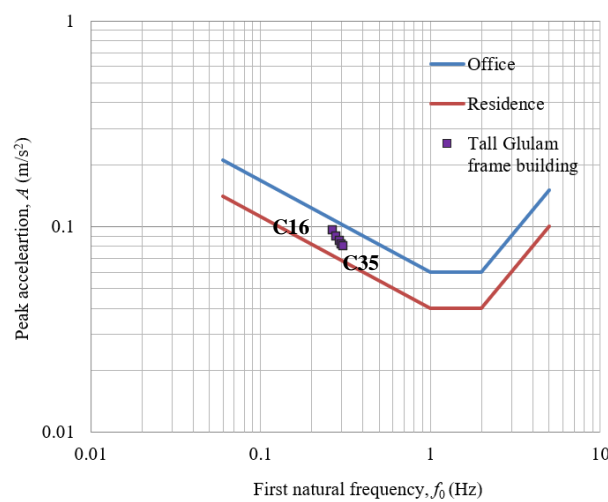
**Figure 6.** The calculated wind-induced peak accelerations with the evaluation curves for residential and office buildings along a horizontal (x or y) direction for a one-year return period by varying the timber material grade for the Glulam members only

The second part is that the new building models for this tall timber building by changing the timber material grade of the CLT members only. Five different timber material grades are adopted, including C16, C20, C24, C30 and C35, respectively. Model A7 uses C16 timber for CLT structural members, Model A8 uses C20 timber, Model A uses C24 timber, Model A9 uses C30 timber, and Model A10 uses C35 timber. Because of the change in the timber material grade, the new equivalent properties of different timber members need be recalculated. After running different numerical models using SAP2000, the fundamental modal frequencies and peak accelerations at the top of the investigated building are obtained and the obtained results are listed in Table 4.

**Table 4.** Parametric study on the peak acceleration by varying the timber material grade of CLT members

Model	Grade	Frequency	Peak acceleration
A7	C16	0.265 Hz	0.096 m/s <sup>2</sup>
A8	C20	0.279 Hz	0.090 m/s <sup>2</sup>
A	C24	0.292 Hz	0.085 m/s <sup>2</sup>
A9	C30	0.298 Hz	0.082 m/s <sup>2</sup>
A10	C35	0.305 Hz	0.080 m/s <sup>2</sup>

Figure 7 illustrates the calculated peak accelerations at the top level of the building versus the fundamental frequency by varying the timber material grade for the CLT members, together with the evaluation curves for residential and office buildings. It can be seen that with the increasing timber grade for the CLT materials only, the fundamental vibrational frequency of the tall Glulam frame building increases and the acceleration at the top level of the building decreases from 0.096 m/s<sup>2</sup> to 0.080 m/s<sup>2</sup>, down by 16.7%. Though the adopted timber material grades are limited, the differences between the obtained results are apparent. This indicates that changing the timber material grade of the CLT members can be an option to enhance the dynamic serviceability performance design of tall timber buildings.



**Figure 7.** The calculated wind-induced peak accelerations with the evaluation curves for residential and office buildings along a horizontal (x or y) direction for a one-year return period by varying the timber material grade for the CLT members only

#### 4.2.2 The effect of the building mass on the peak acceleration of the tall Glulam frame building

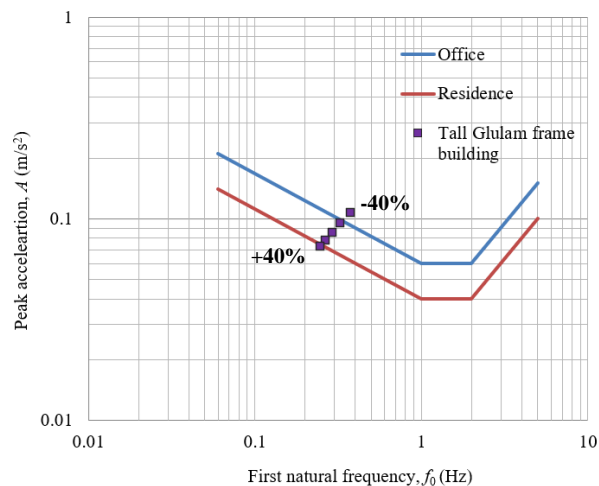
The impact of the building mass should also be included in the tall Glulam frame building. To analyse the effect of the building mass on the peak acceleration of the building, four new models are created. According to Equation (19) below for determining the fundamental frequency and assuming that the generalised stiffness of the building models is not changed, the fundamental frequency  $f$  can be determined, where  $K$  is the generalised stiffness and  $M$  is the generalised mass for the building. Compared with Model A, Model A11 decreases the generalised building mass by 40%, Model A12 decreases the building mass by 20%, Model A13 increases the building mass by 20%, and Model A14 increases the generalised building mass by 40%. The calculated results are listed in Table 5.

$$f = \frac{1}{2\pi} \sqrt{\frac{K}{M}} \quad (19)$$

**Table 5.** Parametric study on the peak acceleration by varying the generalised building mass

Model	Generalised mass	Frequency	Peak acceleration
A11	-40%	0.377 Hz	0.107 m/s <sup>2</sup>
A12	-20%	0.326 Hz	0.095 m/s <sup>2</sup>
A	0%	0.292 Hz	0.085 m/s <sup>2</sup>
A13	+20%	0.267 Hz	0.078 m/s <sup>2</sup>
A14	+40%	0.247 Hz	0.073 m/s <sup>2</sup>

Figure 8 illustrates the calculated peak acceleration at the top level of the investigated building versus the fundamental frequency by varying the generalised building mass by up to  $\pm 40\%$ , together with the evaluation curves for residential and office buildings. It can be seen that with the increasing generalised building mass, the fundamental vibrational frequency of the tall Glulam frame building decreases and the acceleration at the top level of the building also decreases from  $0.107 \text{ m/s}^2$  to  $0.073 \text{ m/s}^2$ , down by 31.8%. Though the adopted building masses are limited, the differences between the obtained results are apparent. This indicates that changing the building mass can be another practical option to enhance the design of tall timber buildings with respect to the global dynamic serviceability performance.



**Figure 8.** The calculated wind-induced peak accelerations with the evaluation curves for residential and office buildings along a horizontal (x or y) direction for a one-year return period by varying the generalised building mass

#### 4.2.3 The effect of the building stiffness on the peak acceleration of the building

To study the effect of the building stiffness on the peak acceleration of the investigated tall Glulam frame building, four new models are created. Compared with Model A, Model A15 decreases the generalised building stiffness by 40%, Model A16 decreases the building stiffness by 20%, Model A17 increases the building stiffness by 20%, and Model A18 increases the generalised building stiffness by 40%. Utilising Equation (19) for determining the fundamental frequency and assuming that the generalised mass of the building models is not changed, the fundamental frequency  $f$  can be determined, and the calculated results are listed in Table 6.

**Table 6.** Parametric study on the peak acceleration by varying the generalised building stiffness

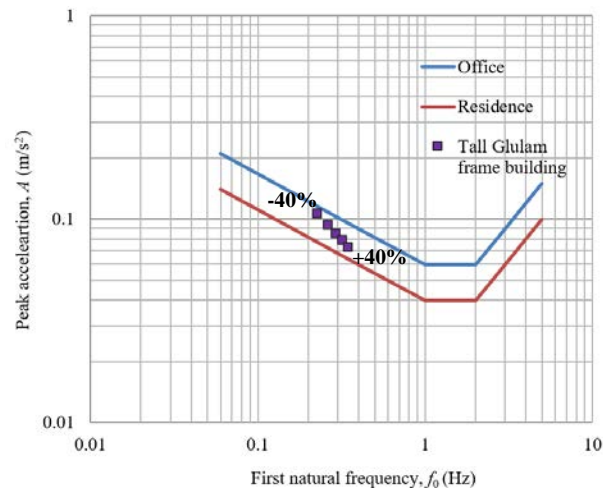
Model	Generalised stiffness	Frequency	Peak acceleration
A15	-40%	0.226 Hz	$0.107 \text{ m/s}^2$
A16	-20%	0.261 Hz	$0.094 \text{ m/s}^2$
A	0%	0.292 Hz	$0.085 \text{ m/s}^2$
A17	+20%	0.320 Hz	$0.079 \text{ m/s}^2$
A18	+40%	0.345 Hz	$0.073 \text{ m/s}^2$

Figure 9 illustrates the calculated peak acceleration at the top level of the investigated building versus the fundamental frequency by varying the generalised building stiffness by up to  $\pm 40\%$ , together with the evaluation curves for residential and office buildings.

It can be seen that with the increasing building stiffness, the fundamental vibrational frequency of the tall Glulam frame building increases, but the acceleration at the top level of the building decreases from  $0.107 \text{ m/s}^2$  to  $0.073 \text{ m/s}^2$ , down by 31.8%. Though the adopted building stiffnesses are limited, the differences between the obtained results are noticeable. This indicates that changing building stiffness can be another practical option to help optimise the design of tall timber buildings with respect to the global dynamic serviceability performance.

To sum up, increasing the timber material grade for the CLT members, the generalised building mass and the generalised building stiffness can all be adopted to lower the peak acceleration at the top level of the investigated Glulam frame building so as to reduce the human perception to the wind induced vibrations with respect to the peak acceleration. However, other criteria under ULS and SLS should also be verified at the same time. Because

the present finite element models do not consider the effects of connections for the CLT members, the actual peak accelerations could be worse.



**Figure 9.** The calculated wind-induced peak accelerations with the evaluation curves for residential and office buildings along a horizontal (x or y) direction for one-year return period by varying the generalised building stiffness

## 5. Conclusions

The global serviceability behaviours of the tall Glulam frame building has been numerically simulated in this research. The first three lowest vibrational frequencies, modes and shapes, together with the static storey and global horizontal displacements have been obtained and analysed. The first three vibrational modes of this tall building are similar to those primarily considered by structural engineers when designing tall reinforced concrete or steel buildings.

The fundamental frequency for this tall Glulam frame timber building has been analysed as 0.292 Hz which is 66.7% of the estimated value or 33.3% smaller in accordance to the Eurocodes and is 44.3% smaller than the recommended by Reynolds et al. At this moment, however, there are no restrictions on the fundamental frequency in the current Eurocodes, indicating that the obtained fundamental frequencies of this building is still acceptable.

Without considering the contributions of connections, shear deformations and the stiffness of internal wall, the maximum horizontal displacement at the top level of this tall Glulam frame building has been numerically determined as 104.36 mm, which is nearly half of the design limit to the Eurocodes. The maximum static storey displacement for this building is 4.096 mm, which is 58.5% of the design limit to the Eurocodes.

According to ISO 10137, the horizontal peak accelerations at the top level of the investigated Glulam frame building is determined and analysed. Without considering the flexibility effects of connections, the wind-induced peak accelerations for the building satisfy the human perception requirements by ISO 10137. Based on the parameter studies on this building, increasing the timber material grade for the CLT members, the generalised building mass and the generalised building stiffness can all be adopted to lower the peak accelerations at the top level of the building so as to reduce the human perceptions to the wind induced vibrations with respect to the peak acceleration.

## 6. Acknowledgements

This research is financially supported by School of Computing, Engineering and Built Environment, Glasgow Caledonian University, Glasgow G4 0BA, Scotland, UK, through the School PhD Scholarship and this is greatly appreciated.

## 7. References

- [1] Palmer S. Sustainable homes: Timber frame housing. The Palmer Partnership. 2000.
- [2] Buchanan AH. Can timber buildings help reduce global CO<sub>2</sub> emissions? In: Proceedings of the 2006 World Conference on Timber Engineering. Portland, Oregon, USA. 2006.

- [3] Dunphy S. Buildings made out of timber could become an important global carbon sink. *European Scientist*. URL: <https://www.europeanscientist.com/en/environment/buildings-made-out-of-timber-could-become-an-important-global-carbon-sink/> (accessed on 08<sup>th</sup> February 2021).
- [4] Brock commons – Tallwood house. Available online: <https://vancouver.housing.ubc.ca/residences/brock-commons/> (accessed on 08<sup>th</sup> February 2021).
- [5] HoHo Vienna: the first city office built of timber in Vienna. Available online: <https://www.buildup.eu/en/ractices/cases/hohovienna-first-city-office-built-timber-vienna> (accessed on 12<sup>th</sup> March 2021).
- [6] Mjöså Tower: World's tallest wood building. Available online: <https://www.metsawood.com/global/news-media/videos/Pages/Mjosa-Tower-worlds-tallest-wood-building.aspx> (accessed on 12<sup>th</sup> March 2021).
- [7] Buchanan AH. The challenges for designers of tall timber buildings. In: *Proceedings of the 2016 World Conference on Timber Engineering*. Quebec City, Canada. 2016.
- [8] Abrahamsen R, Bjertnæs MA, Bouillot J, Brank B, Cabaton L, Crocetti R, Flamand O, Garains F, Gavric I, Germain O, Hahusseau L, Hameury S, Johansson M, Johansson T, Ao WK, Kuren B, Landel P, Linderholt A, Malo K, Manthey M, Nāvik P, Pavic A, Perez F, Rönquist A, Stamatopoulos H, Sustersic I, Tulebekova S. Dynamic response of tall timber buildings under service load – the Tynattb research program. XI International Conference on Structural Dynamics. Athens, Greece. 22–24 June 2020.
- [9] Aloisio A, Pasca DP, Tomasi R, Fragiacomio M. Dynamic identification and model updating of an eight-storey CLT building. *Engineering Structures*. 2020; 213, 110593.
- [10] Edskär I, Lidelöv H. Dynamic properties of cross-laminated timber and timber truss building systems. *Engineering Structures*. 2019; 186:525-535.
- [11] Bjertnæs MA, Malo KA. Wind-induced motions of “Treet” – a 14-storey timber residential building in Norway. In: *Proceedings of the 2014 World Conference on Timber Engineering*. Quebec City, Canada. 2014.
- [12] Johansson M, Linderholt A, Jarnerö K, Landel P. Tall timber buildings – a preliminary study of wind-induced vibrations of a 22-storey building. In: *Proceedings of the 2016 World Conference on Timber Engineering*. Vienna, Austria. 2016.
- [13] BSI. BS EN 1991-1-4:2005+A1:2010: Eurocode 1: Actions on structures – Part 1-4: General actions – Wind actions. British Standards Institution (BSI). London, UK. 2005.
- [14] ISO. ISO 10137 Bases for design of structures – Serviceability of buildings and walkways against vibration. International Organization for Standardization (ISO), Geneva, Switzerland, 2007.
- [15] CSI. SAP2000 Version 18.2 [Computer software]. Computers and Structures, Inc., Berkeley, CA, USA. 2016.
- [16] BSI. BS EN 14080: Timber structures. Glued laminated timber and glued solid timber – Requirements. British Standards Institution (BSI), London, UK. 2013.
- [17] BSI. BS EN 338: Structural timber – Strength class. British Standards Institution (BSI), London, UK. 2016.
- [18] Yasumura M, Kobayashi K, Okabe M, Miyake T, Matsumoto K. Full-scale tests and numerical analysis of low-rise CLT structures under lateral loading. *Journal of Structural Engineering*. 2016; 142(4): E4015007.
- [19] Gagnon S, Pirvu C. CLT Handbook: Cross-Laminated Timber. Canadian Edition, FPInnovations. 2011.
- [20] Wallner-Novak M, Koppelhuber J, Pock K. Cross-laminated timber structural design – Basic design and engineering principles according to Eurocode. ProHolz Austria, Vienna. 2013.
- [21] BSI. BS EN 1995-1-1:2004 + A1:2008 Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings. British Standards Institution (BSI), London, UK. 2004.
- [22] BSI. NA to BS EN 1995-1-1:2004 + A1:2008: UK national annex to Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings. British Standards Institution (BSI), London, UK. 2004.
- [23] BSI. BS EN 1991-1-1 Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings. British Standards Institution (BSI), London, UK. 2002.
- [24] BSI. NA to BS EN 1991-1-1 UK National annex to Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings. British Standards Institution (BSI), London, UK. 2002.
- [25] BSI. BS EN 1990:2002 + A1:2005 Eurocode – Basic of structural design. British Standards Institution (BSI), London, UK. 2002.
- [26] BSI. NA to BS EN 1990: 2002+A1: 2005 UK national annex for Eurocode – Basis of structural design. British Standards Institution (BSI), London, UK. 2002.
- [27] Reynolds T, Feldmann A, Ramage M, Chang WS, Harris R, Dietsch P. Design parameters for lateral vibration of multi-storey timber buildings, In: *International Network on Timber Engineering Research*. Graz, Austria. 2016.
- [28] Gulvanessian H, Calgaro JA, Holicky M. Designers' guide to EN 1990 Eurocode: Basis of structural design. Thomas Telford, London, UK. 2012.
- [29] BSI. NA to BS EN 1991-1-4:2005 + A1:2010: UK national annex to Eurocode 1: Actions on structures – Part 1-4: General actions – Wind actions. BSI, London, UK. 2005.

- [30] CNR. CNR DT 207/2008 Guide for the assessment of wind actions and effects on structures. Advisory Committee on Technical Recommendations for Construction, Consiglio Nazionale delle Ricerche (CNR), Roma, Italy. 2010.



© 2021 by the author(s). This work is licensed under a [Creative Commons Attribution 4.0 International License](http://creativecommons.org/licenses/by/4.0/) (<http://creativecommons.org/licenses/by/4.0/>). Authors retain copyright of their work, with first publication rights granted to Tech Reviews Ltd.