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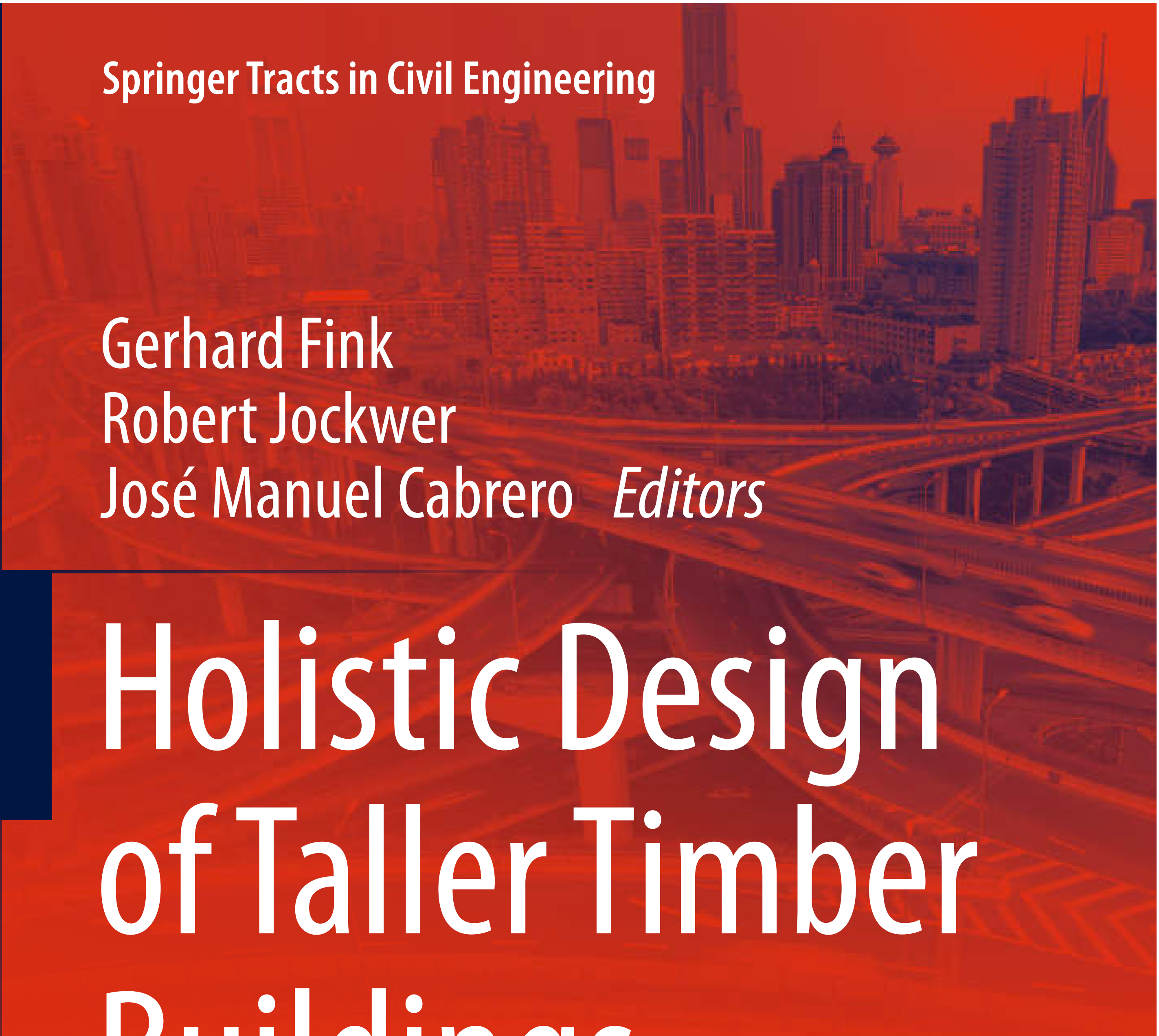

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Holistic Design of Taller Timber Buildings

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Contents

Vibration-Based Wind Design Provisions for Tall Timber Buildings

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1 Introduction

It is well acknowledged that as the height of a building increases, wind forces tend to become the controlling design loads. In tall, slender and flexible structures, wind-induced vibration serviceability is more critical than in low-rise buildings, where strength is usually the governing design criterion. Using engineered timber panels, tall mass timber buildings have reached heights comparable to those of mid-rise concrete and steel buildings. Therefore, wind-induced vibration needs attention even in mid-rise buildings when timber is used as a structural material for lateral load resistance. It has been shown that in timber buildings as low as seven stories wind-induced vibrations can be a governing design criterion [1]. Due to the relatively high strength-to-weight ratio of wood, timber buildings tend to be lighter and more flexible compared to their reinforced concrete counterparts. Typically, timber structures possess one-third the stiffness of concrete buildings and weigh one-fourth as much. This, in turn, means that they are more prone to

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wind-induced vibrations that might cause discomfort to the occupants. Excessive wind-induced vibrations impair their long-term safety and functionality, which lowers their market value and may have an impact on the comfort and health of occupants. For these reasons, serviceability issues related to wind-induced vibration are considered a hindering factor for the design of tall timber buildings.

Compared to the ultimate limit states that define collapse or other forms of catastrophic failure of the structures, serviceability limit states define a loss of comfort or functional performance of the building. They establish a level of quality of the building and are often negotiated by the investor and the contractor to meet the expected functionality of the building. Its limitation may be set by the occupants' perception or technical requirements of the installed building elements or machines. Since serviceability limit states are not directly related to safety, professional committees and code bodies seem reluctant to codify serviceability issues rigidly. This reluctance is probably due to the different opinions on the purpose of building codes: protection for life safety or the establishment of complete minimum design standards. However, the fact that serviceability limit states are usually not codified should not diminish their importance. While safety is usually not an issue in this topic, the economic consequences can be substantial, especially when they are a part of contractual requirements set by the investors.

Whereas the research on the seismic response of timber buildings has progressed at a high pace, wind performance of mid and high-rise timber buildings has been studied more scarcely. There is no specific reported cases on wind-related human discomfort in tall timber buildings, mainly because there is no significant sample of such buildings for a meaningful statistical evaluation. Therefore, the wind modelling and the comfort criteria originate from past studies on other structural systems, see [2]. However, the lack of specific studies on timber buildings does not affect comfort criteria limits, which depend on the interaction between human perception and building dynamics and are not directly related to the building material.

2 Vibration Response Analysis Under Wind Excitation

Comfort criteria are based on the human perception of vibration, which is, due to substantial variation in individual physiological and psychological responses, very difficult to quantify. It is not fully understood how different quantities of motion (such as displacement, velocity, acceleration, and their derivatives) contribute to the perception of motion, however, the current comfort criteria are assessed through acceleration curves. These base curves represent either the threshold of perception of motion of some percentage of people or the limit for probable adverse comments by the occupants [3,4]. They depend on the vibration frequency of the structural system and the orientation of the vibration relative to the human body axes. Firstly, a performance indicator (e.g. running root-mean-square, peak acceleration) is computed based on the building's properties and the assumed wind loads. This indicator is then compared to the base perception curves to evaluate the system's performance. Some of the most relevant standards containing serviceability criteria related to motion perception of the occupants are hereby listed and reported in Fig. 1:

- ISO 10137 (2007) [3]. The standard recommends the serviceability criteria against the building's vibrations. Annex D provides a method for evaluating the human perception of wind-induced motions in buildings, giving acceptable limits in terms of peak acceleration, at the natural frequency, in the principal direction (along-wind, cross-wind, torsion) of the building. The peak acceleration should be calculated for wind speed with a one-year return period. The criteria is based on assuming peak = 3.5*RMS
- ISO 6897 (1984) [4]. The standard, based on [5]. covers building, whose frequency is 0.063 Hz-1 Hz, gives limit values of root mean square (RMS) accelerations for buildings used for general purposes. The limits are based on the wind levels with a five-year return period.
- AIJ (2004) [6]. The Japanese guidelines provide five curves: H-10, H-30, H-50, H-70 and H-90, where the number of each curve indicates the perception probability, expressed as a percentage of people who can perceive the given vibration level. The guidelines recommend no specific limit, which is to be decided by the owner and the designer.
- RWDI/BLWTL industry criterion [7]. A widely used industry practice, especially in North America, limits the 10-year return period peak acceleration to below 18 milli-g. While not formalized in a standard, this criterion is often adopted in practice.

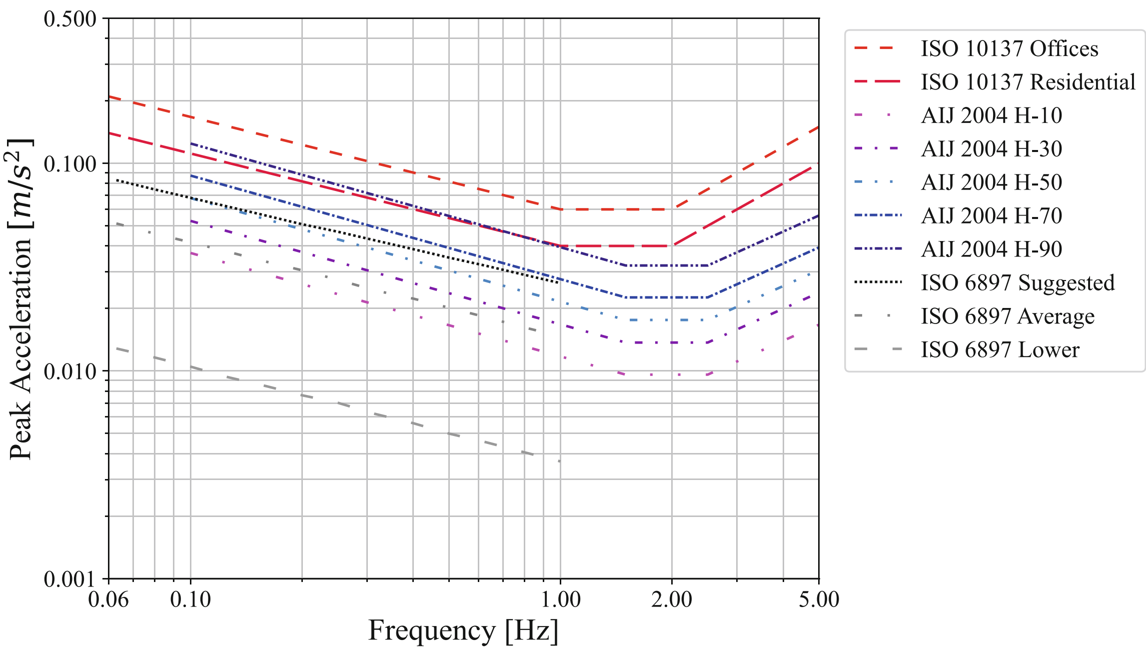

Fig. 1. Comparison between different perception curves (RMS values from ISO 6897 are multiplied by 3.5 to be comparable with peak acceleration limits).

The dynamic response of tall buildings to a wind load is a complex phenomenon, as the wind significantly varies in velocity and direction over time and space. To achieve simplification, building codes treat wind loads as quasi-static loads. The gust effect is also included as a turbulence factor added to the quasi-static component for high slender structures. Furthermore, the effect of the wind on a building will be affected by its exposure, the roughness of the terrain around it as well as the shape and height of the building. The gust load factor approach originates from the work of Davenport [8] and is a simplified frequency-domain method in which the wind load's standard deviation and the dynamic response's amplitude are multiplied by a peak factor to obtain the peak response. The standard deviation of the response and a peak factor are obtained based on random vibrations theory. Because of its simplicity, the gust factor method has received widespread acceptance worldwide and is employed in wind loading codes and standards in almost all major countries. According to the method in Annex B of EN 1991-1-4, for instance, the characteristic wind-induced acceleration, in the along-wind direction, for a point at height z, is calculated as:

$$a_{\text{peak}}(z) \;=\; K_p\, b_b\, h_b\, q_{z,\text{ref}}\, c_{fw}\, K\, \Phi_{1,x}(z)\, \frac{1}{M_1}\, R \tag{1}$$

$a_{\text{peak}}(z)$ Peak acceleration response (m/s^2) at height z
K_p Peak factor
b_b Building width
h_b Building height
$q_{z,\text{ref}}$ Reference wind load (N/m^2) at height z_{ref}
c_{fw} Force coefficient
K Dimensionless coefficient that scales turbulence effects
$\Phi_{1,x}(z)$ First-mode shape in the wind (cross-wind) direction, evaluated at height z
M_1 Modal mass of the first mode
R Resonance response factor

The peak response obtained can then be compared with peak acceleration criteria, such as ISO 10137 [9]. However, the response of buildings subjected to wind loads can be estimated more precisely with time-domain analysis using in-situ wind measurements or wind tunnel experiments, but also with methods combining spectral analysis and time-domain analysis [10–12]. The wind is described in a frequency-domain wind spectrum, and then a time series is generated from the spectrum. The dynamic response of the structure can be computed through numerical integration of the modal equation of motion. It should be mentioned that wind tunnel testing is recommended for mass-timber structures that are more than 20 stories tall, flexible, and have complex shapes, and are subject to wake buffeting from upwind buildings or channelling effects, and are susceptible to aeroelastic phenomena.

For calculating acceleration according to EN 1991-1-4 and checking it against the base curves, estimation of the natural frequency is necessary. Annex F of EN 1991-1-4 offers a simplified empirical equation for the fundamental frequency in Hertz

$$f_1 = 46/h \tag{2}$$

where h is the height of the building in meters. This equation can give a quick rough estimate, however, it can not be used for improving the design of the building, since the only variable is the height of the building. Reynolds et al. [13] compare the equation to ambient vibration test results for eleven multi-storey timber buildings in Europe, and find that it is a reasonable predictor of the fundamental frequency, although $f_1 = 55/h$ is a better fit to the measured data.

Designers of mid- and high-rise timber buildings often develop FE models to estimate their modal properties. Several research works have been carried out to find better modelling techniques for accurate estimation of modal properties of timber buildings (e.g. [14–19]). The main findings are that the nonstructural elements (such as façade, partition walls, plasterboards) can significantly increase the stiffness of the building (although this may not outweigh the effect of their added mass on the fundamental frequency) and that the type of construction (platform frame or balloon frame) importantly defines the stiffness properties of the building. The research is yet inconclusive about the influence of the connections in CLT buildings, however, it seems that soundproofing bedding under cross-laminated walls is an important factor in determining the stiffness of the connections. In glulam frame buildings, the stiffness and damping of the glulam connections can influence the modal properties [17,20]. It is important to note that modal properties of timber buildings vary seasonally, mainly due to environmental factors such as moisture content [21].

Most of the research work carried out during recent years deals with numerical studies where different structural systems (e.g. post-and-beams, CLT, etc.) are analysed throughout case studies and parametric analyses [12,22–32]. Specifically, Johansson et al. [22] studied the response of two archetypes (i.e. CLT buildings, glulam post-and-beam with a concrete shaft) to wind-induced acceleration according to EN 1991-1-4 and compared the results of the simplified analytical calculations with the limits of ISO 10137. They extended the analysis to a 48 m (16 storeys) high building where they studied the effect of doubling and tripling the mass, stiffness and damping ratio. In [23], the authors analysed 22-storey structures having an internal CLT core and a post-and-beam structure at the perimeter. They modelled the structure with FE software to get more reliable results regarding natural frequencies and mode shapes. Edskär and Lindelöw [24] performed a parametric analysis on the FE model of a CLT structure varying the footprint, height, damping ratio, wall stiffness, wall density and additional surface loads and studied the influence of these parameters on the dynamic response of the building. In [25], the authors extended the parametric analyses to a post and beam type of structure and compare it to the response of a

CLT one. In the same direction, Zhao et al. [26,27] performed parametric studies on the peak accelerations of CLT and glulam frames buildings, respectively. The structures were assumed to be located in Glasgow and have 30 storeys. Furthermore, the varied parameters were the timber material properties and building masses. In these papers, the accelerations were also calculated according to the Eurocodes, and the response is evaluated concerning ISO 10137 limits. Landel et al. [28] compared four procedures to evaluate the along wind accelerations on four existing tall timber buildings, highlighting high variations between the different codes. Cao and Stamatopoulos [12] delivered a numerical investigation on the response of moment-resisting frames subject to wind loads. They performed over one million simulations on planar frames where several parameters (e.g. floor height, floor number, beam stiffness) were varied. The response of the planar frames was evaluated with the simplified gust approach, but also performing time-domain analyses explicitly considering the time series of the wind force. A quite interesting finding was that the gust factor approach might underestimate the response for frames with up to 10–12 floors while overestimating the accelerations for frames with more than 10–12 storeys [12]. Bezabeh et al. [29] examined the dynamic response and serviceability performance of five case study tall mass-timber buildings varying in height (10-, 15-, 20-, 30-, and 40-story). Bezabeh et al. [10,11] proposed a probabilistic procedure to assess the serviceability performance of tall mass-timber buildings, applying the complete framework to a case study consisting of a 102-m tall building. The framework incorporated uncertainties at each step of the wind-loading chain. The design process consisted of a preliminary strength design using building code provisions. Then serviceability checks were performed using wind loads obtained from aerodynamic wind tunnel tests. Finally, the detailed probabilistic performance assessment was performed with structural reliability analysis using Monte Carlo sampling to propagate the uncertainties through the wind loading chain. The results from reliability analysis were used to develop fragility curves for wind vulnerability estimations. Lazzarini et al. [30] studied the comfort assessment of the 18-storey “Mjøstårnet” building in Norway, applying computational fluid dynamic (CFD) analyses to simulate the wind flow around the building. The results of the CFD analysis are then used to extrapolate detailed pressure data, which is applied to a generalized model and a reduced model to obtain accurate evaluations of wind-induced motions. Wind-induced vibrations in the across-wind directions were particularly strong, which is not captured by the current standard, indicating the importance of applying fluid dynamic analyses. Kurent et al. [1] performed a serviceability check of two timber and one hybrid timber-concrete buildings and showed that checking only the first mode of vibration is not enough, since the second mode can sometimes be more critical.

3 Conclusions

Wind-induced vibration governs the serviceability design of mid-and high-rise timber buildings at lower heights than it does for comparable concrete structures because the high strength-to-weight ratio of wood leads to lower mass and greater flexibility. Although comfort criteria such as ISO 10137, ISO 6897 and the AIJ guidelines are material-agnostic, their verification still hinges on an accurate prediction of peak accelerations; the review presented here shows that the traditional gust factor approach embedded in EN 1991-1-4 can be adequate only when its inputs, the first two natural frequencies, modal masses and damping ratios, are themselves reliable. Empirical height-frequency formulas (e.g. $f_1 = 46/h$) have yet to be validated for timber and therefore carry large uncertainty, while finite-element predictions remain sensitive to the stiffness contribution of non-structural components, connection bedding and seasonal moisture variation. Case-study simulations confirm that doubling mass or stiffness can halve the peak acceleration, but also reveal that higher modes, particularly the second bending mode, may control comfort more than the fundamental one [1]. Parametric studies further indicate that the gust-factor method may underestimate accelerations for 10- to 12-storey frames and overestimate them for taller systems, suggesting that time-domain analyses or hybrid spectral/time techniques should become routine for slender timber towers [12]. The scarcity of full-scale monitoring data remains the principal obstacle to refining both simplified and sophisticated models; without such data it is impossible to quantify the true damping achievable through façade friction, interior fit-out or supplemental devices, or to calibrate probabilistic fragility curves that capture the combined effect of wind variability and modelling uncertainty.

In practical terms, designers should (i) develop FE models that include sheathing, partitions and realistic connection stiffness, (ii) evaluate at least the first two lateral modes in both along- and cross-wind directions, (iii) supplement code-based gust calculations with time-domain checks when predicted peak accelerations approach 80% of the relevant perception limits, iv) consider wind tunnel testing for mass-timber buildings that are more than 20 stories tall, flexible, and have complex shapes, and are subject to wake buffeting from upwind buildings or channelling effects, and are susceptible to aeroelastic phenomena, and (iv) consider tuned mass or viscous dampers early in the design when mass or stiffness increases conflict with sustainability targets. On the research side, priority should be given to long-term dynamic monitoring of completed mass-timber towers coupled with in-situ wind measurements, to wind-tunnel databases that cover the distinctive geometries of timber bracing and cores, and to probabilistic frameworks that propagate material, connection, and environmental uncertainties through to serviceability risk metrics.

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