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COMPARATIVE STUDY OF WIND-INDUCED ACCELERATIONS IN TALL TIMBER BUILDINGS ACCORDING TO FOUR METHODS

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ABSTRACT: The height and the market share of multi-story timber buildings are both rising. During the last two decades, the Architectural and Engineering Construction industry has developed new reliable solutions to provide strength, structural integrity, fire safety and robustness for timber structures used in the mid- and high-rise sectors.
According to long-time survey and lab experiments, motion sickness and sopite syndrome are the main adverse effects on the occupants of a wind sensitive building. For tall timber buildings, wind-induced vibrations seem to be a new critical design aspect at much lower heights than for traditional steel-concrete buildings. To guarantee good comfort, the horizontal accelerations at the top of tall timber buildings must be limited. Two methods in the Eurocode for wind actions (EN1991-1-4), procedure 1 in Annex B (EC-B) and procedure 2 in Annex C (EC-C), provide formulas to estimate the along-wind accelerations. The Swedish code advises to follow a method specified in the National Annex to the Eurocode (EKS) and the American ASCE 7-2016 recommend another method.
This study gives an overview on the background of the different methods for the evaluation of along-wind accelerations for buildings. Estimated accelerations of several tall timber buildings evaluated according to the different methods are compared and discussed. The scatter of the accelerations estimated with different codes is big and increases the design uncertainty of wind induced response at the top of tall timber buildings.

KEYWORDS: Tall timber buildings, serviceability, wind loads, dynamical response, along-wind peak acceleration, wind-induced vibrations, comfort, building code

1 INTRODUCTION

The response of a building excited by wind is not only dependent on that building´s dynamical properties. The complex behaviour of wind sensitive structures also depends on the low-frequent turbulent properties of the wind excitation. Davenport´s wind loading chain simplifies the understanding of the wind effects on constructions and his gust loading factor method is still used in most codes [1]. The Eurocode on wind loads EN 1991-1-4:2005 [2] proposes two informative methods to calculate the along-wind acceleration for human comfort. The first one in the annex B (EC-B) which is based on research work made by Solari and some explanations can be found in his papers [3]. The second one in the annex C (EC-C) which is based on studies presented by Hansen and Krenk [4]. In Sweden, the recommended method is presented in the national Swedish annex, EKS [5] and some background and assumptions for its derivation can be found in Handas papers [6-7]. The method described in the American ASCE 7-2016 [8] is based on the gust-effect factor and inspired by Solari and Kareem's studies [9].

Serviceability design calculations for wind-induced vibration of tall timber buildings have been published in research papers during the last decade. Results for calculated top accelerations have been presented in relation to the standard thresholds. Impacts on different structural design changes, usually dealing with mass, stiffness and damping of the timber structure or impact of the non-structural material in the building, have been analysed and discussed. The relevance of the thresholds for accelerations set in standards is also crucial for the design and for the inhabitant's comfort [10-11] but is not assessed in the present paper.
Malo and Abrahamsen [12-13] presented acceleration results for the design of two tall timber buildings based on the EC-C method, which is the recommended method in Norway. The EC-B method has been used in several parameter studies to optimize the design of conceptual tall timber buildings up to 30 storeys [14-15]. The Swedish EKS method has been used for comfort design investigations on CLT buildings up to 22 storeys [16-17]. The Canadian method has been used and compared to wind tunnel tests for analyses of wind excessive motion of tall mass timber building concepts up to 40 levels [18].

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Comparative studies of different building codes on the along-wind acceleration have been performed mainly for traditional tall buildings with steel and concrete structures but not for lighter timber buildings. Kareem and his colleagues compared the wind effects estimated from dozens of national codes using 200-m tall building cases [1, 19].
Hence, no comparison on the along-wind acceleration of different codes have been performed for tall timber buildings which are in general lighter and less stiff than buildings in steel and concrete. Whereas the impact of structural dynamical parameters on comfort due to wind excitation has been studied, differences between several methods on the estimation of the along peak acceleration might be interesting to better understand the uncertainty of the models and the background assumptions.

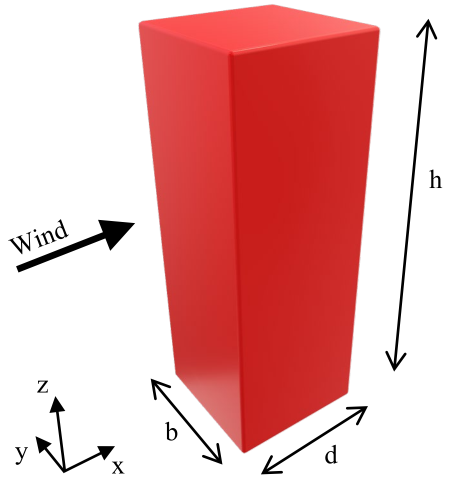


Figure 1: *Graphical representation of the building, the wind direction and the coordinate system*

2 TOP ACCELERATION BASED ON DIFFERENT BUILDING CODES

The methods from the building codes used in this study are all based on similar input data, but the models vary. Structurally, the building is modelled as a cantilevered beam, clamped in the ground, and assumed to vibrate in one single transversal eigenmode with the natural frequency $n_{1,x}$, the structural damping ratio ξ, the mode shape $\phi_{1,x}$ which is a function of the elevation z, and the modal mass distribution. The excitation is modelled as a random stationary process of turbulent wind gusts with low frequency. The main building dimensions are the height h, the width b, normal to the wind, and the depth d. The coordinate system is shown in Figure 1. The transversal deflection of the cantilevered building as a function of the altitude is x(z). The second derivative of x(z) with respect to time is the acceleration which is denoted $\ddot{x} = \frac{d^2x}{dt^2}$.

To ensure good comfort and low wind-induced vibration in buildings, the along-wind accelerations should be lower than some threshold. Depending on people's occupation, the building codes recommend either single limits such as 10 or 20 milli-g (100 or 200 mm/s^2) or frequency dependent limits such as in the ISO 6897:1984 in which the standard deviation of the acceleration, also known as the root mean square (r.m.s.) of the acceleration, $\sigma_{\ddot{x}}$ is used [20]. In the ISO 10137:2007 the peak acceleration $\ddot{X}_{max}$ is applied, with increasing limits for decreasing frequency below 1 Hz [21].
The relationship between r.m.s. and peak acceleration is obtain using the peak factor k_p according to:

$$\ddot{X}_{max} = k_p \, \sigma_{\ddot{x}} \qquad (1)$$

The elevation used for the evaluation of comfort acceleration is usually the height from the surrounding ground to the upper side of the occupied top floor.
The derivation of the r.m.s. acceleration is not the same in each code as the assumptions for the dynamical phenomena, the wind velocity and the turbulence vary. The wind velocity profile can be expressed as a power law like in the ASCE, or as a logarithmic law like in the European codes. The wind velocity profile includes variables depending on the terrain exposure such as the roughness and the power exponent. Similarities in the final derivation given in the building code are noticeable and the peak acceleration $\ddot{X}_{max}(z)$ in m/s^2 can be given as a function of the wind pressure q_{zref} in N/m^2, the global peak factor K_p including turbulence, the resonance response factor R, the force factor c_f, the area $b \cdot h$ in m^2 normal to the wind direction, the modal factor K, the first mode shape $\phi_{1,x}(z)$ and its modal mass M_1 in kg:

$$\ddot{X}_{max}(z) = b\,h\;q_{z_{ref}}\,c_f\,K_p\,K\,\phi_{1,x}(z)\,\frac{1}{M_1}\,R \qquad (2)$$

The different parameters according to the four building codes are reported in Table 1. The force coefficient c_f considers the shape of the building and the overall wind action around it, i.e. wind pressures and friction on all the sides. It is an empirical factor in the building codes that has been tuned from wind tunnel results. The force coefficient is the same in the Eurocodes and in the EKS but not in the ASCE code. The wind pressure $q_{z_{ref}}$ at the height z_{ref} is calculated based on the air density ρ and the square of the mean wind speed $v_{m,z_{ref}}$ for the actual return period. The air density is equal to 1.25 kg/m^3 in Europe and 1.225 kg/m^3 in USA and Canada.
For the ultimate state design, the wind velocity is based on a 50-year return period occurrence but for serviceability the return period of event is lower. Recommendations vary between 1- and 10-year return periods and a 5-year return period is chosen in this study. The probability factor c_p of 0.855 is used to estimate the 5-year velocity from the basic 50-year value (either 3-sec gust or 10-min mean), according to the Eurocode and with a probability of annual exceedance p = 1/T = 0.2 with T equal to five years.

Table 1: *Along-wind acceleration main parameters*

	ASCE	EC-B	EC-C	EKS
z_{ref}	$0.6\,h$	$0.6\,h$	$0.6\,h$	h
$q_{z_{ref}}$	$\frac{1}{2}\,\rho\,v_{m,z_{ref}}^2$	$\frac{1}{2}\,\rho\,v_{m,z_{ref}}^2$	$\frac{1}{2}\,\rho\,v_{m,z_{ref}}^2$	$\frac{1}{2}\,\rho\,v_{m,z_{ref}}^2$
c_f	$\frac{h}{60\,b}+1.2833$ for $b \leq h \leq 7\,b$	$c_{f,0}\,\psi_r\,\psi_\lambda$	$c_{f,0}\,\psi_r\,\psi_\lambda$	$c_{f,0}\,\psi_r\,\psi_\lambda$
K_p	$1.7\,k_p\,I_{v,z_{ref}}$	$2\,k_p\,I_{v,z_{ref}}$	$2\,k_p I_{v,z_{ref}}$	$2\,k_p\,I_{v,z_{ref}}$
K	$\frac{1.65^{\hat{\alpha}}}{\hat{\alpha}+\zeta+1}$	$\frac{(2\zeta+1)\left[(\zeta+1)\left[\ln\left(\frac{z_{ref}}{z_0}\right)+0.5\right]-1\right]}{(\zeta+1)^2\ln\left(\frac{z_{ref}}{z_0}\right)}$	$K=\frac{K_y\,K_z}{\phi_{max}}$ $K_y=\phi_{max}=1$ $K_z=\frac{\zeta+8}{6}$ for $1 \leq \zeta \leq 2$	$\frac{3}{2}$
$\phi_{1,x}(z)$	$\left(\frac{z}{h}\right)^{\zeta}$	$\left(\frac{z}{h}\right)^{\zeta}$	$\left(\frac{z}{h}\right)^{\zeta}$	$\left(\frac{z}{h}\right)^{1.5}$
M_1	$\int_0^h m(z)\,\phi_1^2(z)dz$	$M_1=h\,m_{1,x}$ $m_{1,x}=\frac{\int_0^h m(z)\cdot\phi_1^2(z)dz}{\int_0^h \phi_1^2(z)dz}$	$M_1=b\,h\,\mu_e$ $\mu_e=\frac{\int_0^h\int_0^b \mu(y,z)\cdot\phi_1^2(y,z)dydz}{\int_0^h\int_0^b \phi_1^2(y,z)dydz}$	$\int_0^h m(z)\,dz$

z_{ref} reference altitude in m ; q_{zref} wind pressure at z_{ref} in N/m^2 ; ρ air density in kg/m^3 ; $v_{m,zref}$ 5-year mean wind velocity at z_{ref} in m/s ; K_p global peak factor ; k_p peak factor ; $I_{v,zref}$ turbulence intensity at z_{ref} ; R resonance response factor ; c_f force coefficient with $c_{f,0}$, ψ_r, ψ_λ according to the Eurocode ; K modal factor with $\hat{\alpha}$ according to the ASCE ; ζ mode shape exponent ; z_o terrain roughness ; M_1 modal mass in kg ; $m_{1,x}$ along-wind fundamental equivalent mass in kg/m ; μ_e reference mass per unit area in kg/m^2 ; m(z) mean vertical mass in kg/m ; $\phi_{1,x}(z)$ mode shape

The reference wind velocities at 10-meters for the case studies are chosen according to the location of the buildings. As they are situated in different countries, the averaging times for reference wind velocities data are not the same. Conversion of wind speed referenced for different averaging times from hourly mean to other averaging times for the same height and terrain exposure can be done according to ISO 4354:2009 [22]:

$v_{1hour} = v_{10min}/1.05 = v_{3sec}/1.53$

Table 2: *Wind velocity parameters*

	ASCE	EC-B, EC-C and EKS
$v_m(z)$	$\bar{b}\left(\frac{z}{10}\right)^{\bar{\alpha}} c_p\,v_{b,3s}$	$0.19\left(\frac{z_0}{0{,}05}\right)^{0{,}07}\ln\left(\frac{z}{z_0}\right) c_p\,v_{b,10min}$
k_p	$\sqrt{2\,\ln(n_{1,x}\,T)}+\frac{0.577}{\sqrt{2\,\ln(n_{1,x}\,T)}}$	$\sqrt{2\,\ln(\nu\,T)}+\frac{0.6}{\sqrt{2\,\ln(\nu\,T)}}$
$I_v(z)$	$c^*\,\frac{10}{z}$	$\frac{1}{\ln(z/z_0)}$

$v_m(z)$ mean wind velocity function of elevation z in m/sec ; $\bar{b}$, $\bar{\alpha}$, c^*, z_0, ℓ and $\bar{\epsilon}$ depend on the terrain exposure ; c_p probability factor ; $v_{b,3s}$ basic 3-sec mean gust wind speed in m/sec ; $v_{b,10min}$ basic 10-min mean wind speed in m/sec ; $I_v(z)$ turbulence intensity function of elevation z ; $n_{1,x}$ first along-wind natural frequency ; $\nu = n_{1,x}\sqrt{\frac{R^2}{B^2+R^2}}$

The square of the resonant response factor R is generally expressed as a function of a size reduction factor S, an energy factor E and a total damping ratio ξ_{tot}.

The energy factor E represents the amount of energy embedded in the wind gust velocity blowing at a frequency near the first natural frequency of the building. It is estimated from empirical wind power spectral density functions. Building codes use different wind spectra. The ASCE and the Eurocodes rely on the wind spectrum developed by Kaimal in 1972 whereas the Swedish EKS is based on Karman's wind spectrum from 1948 [1].

The size factor S considers the spatial correlation of the wind velocity. At a specific time, the wind blows with different velocities at different places on the swaying structure which reduces the global mean wind load. The size factor is therefore between zero and one and is decomposed in two- or three-dimensional correlations factors also called exponential decay parameters [3,4].

The resonance response factor is inversely proportional to the total damping which is the structural damping in the building plus the aerodynamic damping due to the interaction between the wind flow and the building envelop. Aerodynamic damping is not considered as additional damping in the ASCE method. For a building with extra active or passive damping, the corresponding extra damping ratio should be added to the total damping ratio. The background response factor in the ASCE is denoted Q whereas in B is used in the Eurocodes and the EKS. The parameters for the resonance and the background response factor of the four building codes are presented in Table 3 and complementary parameters are given in Table 4.

__Table 3:__ Resonance and background response main parameters

	ASCE	EC-B	EC-C	EKS
R^2	$2\,\pi\,\frac{E\,S}{\delta_s}$	$\frac{\pi^2}{2}\frac{E\,S}{(\delta_s+\delta_a)}$	$\frac{\pi^2}{2}\frac{E\,S}{(\delta_s+\delta_a)}$	$2\,\pi\,\frac{E\,S}{(\delta_s+\delta_a)}$
E	$\frac{7.47\,N_1}{[1+10.3\,N_1]^{\frac{5}{3}}}$	$\frac{6.8\,f_L}{[1+10.2\,f_L]^{\frac{5}{3}}}$	$\frac{6.8\,f_L}{[1+10.2\,f_L]^{\frac{5}{3}}}$	$\frac{4\,y_C}{[1+70.8\,{y_C}^2]^{\frac{5}{6}}}$
S	$R_h R_b(0.53+0.47R_d)$	$R_h R_b$	$\frac{1}{1+\sqrt{(G_y\cdot\phi_b)^2+(G_z\cdot\phi_h)^2+\left(\frac{2}{\pi}\cdot G_y\cdot\phi_b\cdot G_z\cdot\phi_h\right)^2}}$	$\phi_h\,\phi_b$
δ_a	0	$\frac{c_f\,\rho\,b\,v_{m,z_{ref}}}{2\,n_{1,x}\,m_{1,x}}$	$\frac{c_f\,\rho\,v_{m,z_{ref}}}{2\,n_{1,x}\,\mu_e}$	$\frac{c_f\,\rho\,h\,b\,v_{m,z_{ref}}}{2\,n_{1,x}\,M_1}$
B^2	$\frac{1}{1+0.63\left(\frac{b+h}{L_{z_{ref}}}\right)^{0.63}}$	$\frac{1}{1+0.9\left(\frac{b+h}{L_{z_{ref}}}\right)^{0.63}}$	$\frac{1}{1+1.5\sqrt{\left(\frac{b}{L_{z_{ref}}}\right)^2+\left(\frac{h}{L_{z_{ref}}}\right)^2+\left(\frac{b\,h}{{L_{z_{ref}}}^2}\right)^2}}$	$e^{\left(-0.05\left(\frac{h}{10}\right)+\left(1-\frac{b}{h}\right)\left(0.04+0.01\left(\frac{h}{10}\right)\right)\right)}$
$L_{z_{ref}}$ in m	$\ell\left(\frac{z_{ref}}{10}\right)^{\overline{\in}}$	$300\left(\frac{z_{ref}}{200}\right)^{0.67+0.05\ln(z_0)}$	$300\left(\frac{z_{ref}}{200}\right)^{0.67+0.05\ln(z_0)}$	150

R resonance response factor ; E energy factor ; S global size reduction factor for wind space correlation ; δ_s logarithmic decrement of structural damping equal to $2\,\pi\,\xi$ with ξ the structural viscous damping ratio to critical ; δ_a logarithmic decrement of aerodynamic damping ; N_1, y_C, f_L non-dimensional frequency ; R_h, R_b, R_d, ϕ_h, ϕ_b specific size reduction factor ; G_y, G_z mode shape constants ; B background response factor ; L_{zref} turbulence length scale in m ; z_0, ℓ and $\overline{\in}$ depend on the terrain exposure

__Table 4:__ Complementary parameters for resonance response

ASCE	EC-B	EC-C	EKS
$R_? = \frac{1}{\eta_?} - \frac{1}{2\cdot{\eta_?}^2}(1-e^{-2\cdot\eta_?})$ $R_? = 1$ for $\eta_? = 0$ $\eta_h = 4.6\,n_{1,x}\,h/v_{m,z_{ref}}$ $\eta_b = 4.6\,n_{1,x}\,b/v_{m,z_{ref}}$ $\eta_d = 15.4\,n_{1,x}\,d/v_{m,z_{ref}}$	$R_? = \frac{1}{\eta_?} - \frac{1}{2\cdot{\eta_?}^2}(1-e^{-2\cdot\eta_?})$ $R_? = 1$ for $\eta_? = 0$ $\eta_h = 4.6\,n_{1,x}\,h/v_{m,z_{ref}}$ $\eta_b = 4.6\,n_{1,x}\,b/v_{m,z_{ref}}$	$\phi_h = 11.5\,n_{1,x}\,h/v_{m,z_{ref}}$ $\phi_b = 11.5\,n_{1,x}\,b/v_{m,z_{ref}}$ $G_y = 0.5$ $G_z = \frac{34-7\zeta}{72}$ with $1 \le \zeta \le 2$	$\phi_? = \frac{1}{1+\eta_?}$ $\eta_h = 2\,n_{1,x}\,h/v_{m,z_{ref}}$ $\eta_b = 3.2\,n_{1,x}\,b/v_{m,z_{ref}}$
$N_1 = \frac{n_{1,x}\,L_{z_{ref}}}{v_{m,z_{ref}}}$	$f_L = \frac{n_{1,x}\,L_{z_{ref}}}{v_{m,z_{ref}}}$	$f_L = \frac{n_{1,x}\,L_{z_{ref}}}{v_{m,z_{ref}}}$	$y_C = \frac{n_{1,x}\,L_{z_{ref}}}{v_{m,z_{ref}}}$

$R_?$ and $\phi_?$ specific size reduction factors with ? either h, b or d respectively ; $\eta_?$ specific size reduction variables with ? either h, b or d respectively

3 FIVE TALL TIMBER BUILDINGS

For this comparative study, four real tall timber buildings and one fictive have been investigated. The peak accelerations at the top floor of the five residential buildings have been estimated using the four methods. The buildings are simplified as boxes with the height h, the width normal to the wind b and the depth d. This simplification is done even though some of the buildings have special, irregular shapes that are not treated in the simplified methods of building codes. The wind properties at the locations and the structural parameters of the buildings have been found in research papers or building codes as specified below. The first natural frequency $n_{1,x}$ of each building has been estimated according to Eurocode 1, annex F for tall structures: $n_{1,x} = 46/h$, although this estimation is not recommended for building heights lower than 50 m [2]. According to the research papers or assumptions, the building locations have been assigned to the terrain categories II or III as defined in the Eurocode 1 and they correspond to the exposure categories C and B respectively as defined in the ASCE 7.

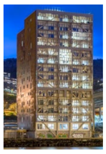
The first building is Origine, located in Québec City in Canada and it consists of a 12 storey balloon-framed CLT structure above a concrete podium of one storey. Most of the information regarding the building has been found in [24]. The CLT walls are up to almost 300 mm thick, about 3-storey high and connected with thick and large steel plates to resist lateral wind loads and earthquakes. When completed in 2018, it was the tallest CLT building in the world. It is wider than tall, i.e. it has a slenderness ratio of height over width lower than one. Thus, the building is not structurally behaving as a cantilevered beam. According to the National Building Code of Canada 2015, the 50-year hourly wind pressure in Québec is 0.41 kPa which yield an hourly wind speed of 25.2 m/s according to NBCC's table C-1 [23]. Using the referenced averaging time factors for wind speed from the ISO 4354:2009 [22] it corresponds to 26.4 m/s of 10-min mean wind speed for the European methods and 38.6 m/s of three-second gust speed in the American method.

The second building is TREET, a 14-storey block in Bergen in Norway, with a three-dimensional truss structure made of large glulam members [12]. Several 8 mm steel plates and plenty of 12 mm diameter dowels are used to connect the columns, diagonals and beams. Three heavy slabs of concrete are placed at the 6th, 11th and top floors increasing the mass of the building and bearing the prefabricated timber modules stacked upon each other.

The third building is the Cultural Centre "SARA", in Skellefteå in Sweden which is planned to be finished by the end of 2021. It is a complex building consisting of several multifunctional parts such as housing art, music, literature, theatre and meeting events with a 20-storey hotel standing in the centre [25]. This study focuses on the 72 m high tower but assumed to be standing alone as a cantilever beam. It is made of prefabricated CLT-modules with two CLT-cores, one on each side of the rectangular plan, stabilizing the building. The CLT-walls of the cores are 300-800 mm thick CLT-walls. Concrete slabs with 300 mm thickness are added on the 19th and 20th levels and on the roof to increase the mass at the building top and reduce the wind-induced vibrations. The mode shape of the tall CLT buildings is assumed here as $(z/h)^{1.3}$.

The fourth building is Mjøstårnet for which structural details have been collected from [13, 26]. It is located in Brumunddal in Norway and it is the highest all timber building in the world since its completion in 2019. The structure consists of massive glulam trusses along the façades as well as internal columns and beams. The cross-sections of the truss elements are up to 1485 x 625 mm^2. The top structural height is 85.4 m according to the CTBUH classification and the level of the top floor is 68.2 m above ground [27]. Between those two altitudes, a pergola ornament made of light hollowed glulam members stands at the top of the tower. The pergolas impact on the dynamical response of the wind-excited building is assumed to be minimal due to low mass and low drag force due to the wind. Therefore, the total height of Mjøstårnet in this study is reduced to 76 m. The finite element model presented by Malo and Abrahamsen estimated the first eigenfrequency in the weak direction to be about 0.37 Hz [13] but the first eigenfrequency used in this study follows the rule of thumb 46/h = 0.54 Hz. Extra mass is also added on the top of the building, the floors 12 to 18 have 300 mm concrete which increases roughly the building density from 90 to 170 kg/m^3.

Table 5: *Wind and structural input parameters for the different buildings*

	ORIGINE Canada	TREET Norway	SARA Sweden	MJØSTÅRNET Norway	TTBtest Case study
Picture of the building					
Building height, h	41.0 m	45.0 m	72.0 m	76.0 m	100.0 m
Evaluation height, z	37.8 m	40.8 m	68.0 m	68.2 m	96.0 m
Width, b	45.6 m	23.0 m	41.0 m	37.0 m	20.0 m
Depth, d	19.5 m	21.0 m	16.0 m	17.0 m	20.0 m
Ref. wind speed, v_b	26.4 m/s	26.0 m/s	22.0 m/s	22.0 m/s	20.0 m/s
Terrain class (EU/US)	III / B	III / B	II / C	II / C	III / B
First natural frequency, $n_{1,x}$	1.12 Hz	1.00 Hz	0.64 Hz	0.54 Hz	0.46 Hz
Struct. damping ratio to critical, ξ	2.0 %	1.8 %	2.3 %	1.9 %	2.0 %
Equivalent mass density	90 kg/m^3	100 kg/m^3	200 kg/m^3 1) 110 kg/m^3	170 kg/m^3 2) 90 kg/m^3	100 kg/m^3

1) above 62 m ; 2) above 51 m

The fifth building is a fictive tall timber building (TTBtest) with simple geometrical and structural values to give the reader an easier understanding of the estimation of the different parameters, the calculation process and the impact on the response. It is 100 m tall, has a building cross section of 20 x 20 m and a bulk building density of 100 kg/m^3 constant along the height. The viscous damping ratio to the critical damping is set to 2.0 %.

4 COMPARISON AND DISCUSSION

4.1 THE ANALYTICAL RESULTS

For each building, the peak accelerations have been estimated using the four different building codes. The results are presented in Figure 2, the main interesting parameters are reported in Table 6 and the complementary parameters in Table 7.

The results and the parameters estimated by the different codes have been compared two by two using the ratio $\frac{\ddot{x}_{largest}}{\ddot{x}_{smallest}}$.

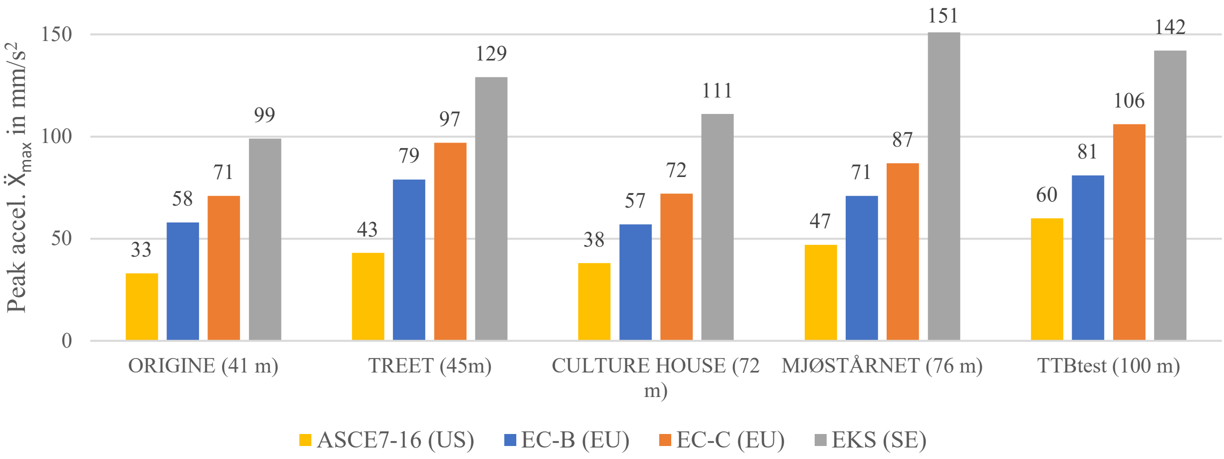


Figure 2: *Along-wind peak acceleration results by buildings and codes*

4.2 RESULT COMPARISONS

The results in Figure 1 show that the American method generates the lowest accelerations and the Swedish method the highest. The ratios between the Swedish results and the ASCE results range between 2.4 and 3.2 with a mean ratio of about 2.9. The largest ratios are for the Mjøstårnet, 3.2, and for the SARA Cultural Centre, 2.9. Both buildings are located in smoother terrain and have extra mass at the top. Differences in wind pressure and building responses are lower at rougher terrain [1] and as mentioned before the Swedish method does not consider non uniform mass distribution but only a mean mass value uniformly distributed along the height.

Between the accelerations assessed with the American and the Swedish methods, the acceleration estimated with the Eurocode method C are slightly higher than the one estimated with the method B, with ratios between 1.2 and 1.3. The comparison between the ASCE and the EC-B method shows varying ratios between 1.4 and 1.8. In a comparative study between major building codes using a fictive 200 m tall building, with a building density of 180 kg/m^3 and located in a terrain exposure III / B, the factor between the estimated accelerations with the EC-B methods were 1.5 compared to the ASCE method [1]. For the same building the difference were 1.8 between the EC-C and the ASCE method. Similar levels of differences are confirmed here for the lower and lighter buildings SARA Cultural Centre and Mjøstårnet located in the same terrain exposure. The ratios between EKS and Eurocode EC-B range between 1.6 and 2.1, with a mean ratio of almost 1.9.

Table 6: *Estimated parameters per building case and predicted with the different codes*

		$\ddot{X}_{max}$ (mm/s^2)	z_{ref} (m)	q_{zref} N/m^2	c_f (-)	K_p (-)	K (-)	M_1 (kg)	K / M_1 (10^{-6}kg^{-1})	$\Phi_{1,x}$ (-)	R (-)	$v_{m,zref}$ (m/s)	k_p (-)	$I_{v,zref}$ (-)
ORIGINE	ASCE	**33**	24,6	211	1,30	1,851	0,50	1093716	0,4583	0,922	0,082	18,6	4,22	0,26
h = 41 m	EC-B	**58**	24,6	287	1,38	1,486	1,50	3281148	0,4572	0,922	0,125	21,4	3,27	0,23
b = 45.6 m	EC-C	**71**	24,6	287	1,38	1,518	1,50	3281148	0,4572	0,922	0,150	21,4	3,34	0,23
z = 37.8 m	EKS	**99**	41	357	1,38	1,358	1,50	3281148	0,4572	0,885	0,195	24	3,34	0,20
TREET	ASCE	**43**	27	214	1,32	1,811	0,50	724500	0,6919	0,907	0,131	19	4,19	0,25
h = 45 m	EC-B	**79**	27	290	1,43	1,493	1,50	2173500	0,6901	0,907	0,197	22	3,36	0,22
b = 23 m	EC-C	**97**	27	290	1,43	1,519	1,50	2173500	0,6901	0,907	0,238	22	3,42	0,22
z = 40.8 m	EKS	**129**	45	359	1,43	1,365	1,50	2173500	0,6901	0,863	0,299	24	3,42	0,20
SARA	ASCE	**38**	43,2	306	1,31	1,087	0,44	1934730	0,2265	0,928	0,141	22,3	4,08	0,16
h = 72 m	EC-B	**57**	43,2	365	1,42	0,950	1,58	6965028	0,2269	0,928	0,187	24,2	3,21	0,15
b = 41 m	EC-C	**72**	43,2	365	1,42	0,972	1,55	6965028	0,2225	0,928	0,235	24,2	3,29	0,15
z = 68 m	EKS	**111**	72	422	1,42	0,912	1,50	6258240	0,2397	0,918	0,312	26	3,32	0,14
MJØSTÅRNET	ASCE	**47**	45,6	311	1,32	1,067	0,50	2331183	0,2148	0,897	0,197	22,5	4,04	0,16
h = 76 m	EC-B	**71**	45,6	370	1,47	0,952	1,50	6993548	0,2145	0,897	0,253	24,3	3,24	0,15
b = 37 m	EC-C	**87**	45,6	370	1,47	0,970	1,50	6993548	0,2145	0,897	0,307	24,3	3,31	0,15
z = 68.2 m	EKS	**151**	76	428	1,47	0,911	1,50	5577108	0,2690	0,850	0,410	26	3,34	0,14
TTBtest	ASCE	**60**	60	189	1,37	1,513	0,41	1000000	0,4064	0,941	0,202	18	4,00	0,22
h = 100 m	EC-B	**81**	60	238	1,47	1,206	1,63	4000000	0,4075	0,941	0,252	20	3,19	0,19
b = 20 m	EC-C	**106**	60	238	1,47	1,239	1,58	4000000	0,3958	0,941	0,328	20	3,28	0,19
z = 96 m	EKS	**142**	100	286	1,47	1,137	1,50	4000000	0,3750	0,941	0,422	21	3,30	0,17

Table 7: *Complementary parameters per building case and predicted with the different codes*

		S	E	ξ_{tot}	B^2	N_1, f_L, y_C	L_{zref}	R_h, Φ_h	R_b, Φ_b	η_h	η_b
		(-)	(-)	(-)	(-)	(-)	(m)	(-)	(-)	(-)	(-)
ORIGINE	ASCE	0,004	0,038	2,0%	0,004	7,9	132	0,084	0,076	11,4	12,7
h = 41 m	EC-B	0,008	0,051	2,7%	0,008	4,4	84	0,096	0,087	9,9	11,0
b = 45.6 m	EC-C	0,012	0,051	2,7%	0,012	4,4	84				
z = 37.8 m	EKS	0,026	0,031	2,2%	0,026	7,0	150	0,207	0,128	3,8	6,8
TREET	ASCE	0,008	0,040	1,8%	0,008	7,3	136	0,086	0,161	11,1	5,7
h = 45 m	EC-B	0,018	0,053	2,5%	0,018	4,1	88	0,099	0,183	9,6	4,9
b = 23 m	EC-C	0,026	0,053	2,5%	0,026	4,1	88				
z = 40.8 m	EKS	0,052	0,034	2,0%	0,052	6,3	150	0,210	0,246	3,8	3,1
SARA	ASCE	0,010	0,046	2,3%	0,010	5,8	204	0,100	0,168	9,5	5,4
h = 72 m	EC-B	0,019	0,058	3,2%	0,019	3,6	135	0,108	0,180	8,8	5,0
b = 41 m	EC-C	0,031	0,058	3,2%	0,031	3,6	135				
z = 68 m	EKS	0,052	0,048	2,6%	0,052	3,7	150	0,220	0,237	3,5	3,2
MJØSTÅRNET	ASCE	0,014	0,051	1,9%	0,014	4,9	206	0,112	0,215	8,4	4,1
h = 76 m	EC-B	0,028	0,064	2,8%	0,028	3,1	139	0,121	0,230	7,8	3,8
b = 37 m	EC-C	0,041	0,064	2,8%	0,041	3,1	139				
z = 68.2 m	EKS	0,070	0,054	2,3%	0,070	3,1	150	0,242	0,290	3,1	2,4
TTBtest	ASCE	0,015	0,053	2,0%	0,015	4,6	177	0,080	0,330	12,0	2,4
h = 100 m	EC-B	0,031	0,060	2,9%	0,031	3,4	144	0,088	0,356	10,8	2,2
b = 20 m	EC-C	0,053	0,060	2,9%	0,053	3,4	144				
z = 96 m	EKS	0,079	0,053	2,3%	0,079	3,2	150	0,189	0,421	4,3	1,4

4.3 DISCUSSIONS AND FURTHER ANALYSES

To better illustrate and understand the huge range of acceleration ratios between the codes, the recommended thresholds for buildings with different types of occupation can be compared. The peak acceleration threshold values for office buildings according to the ISO 10137:2007 [21], see Figure 3, are 1.5 times higher than for residential buildings.

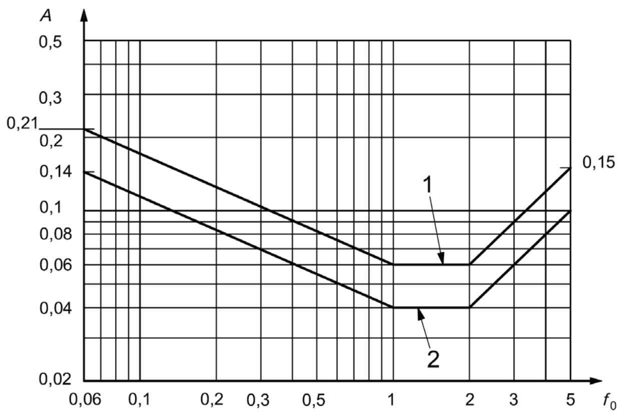


Figure 3: *Peak acceleration thresholds, A, in m/s² as a function of the first natural building frequency, f_0, in Hz for (1) office, (2) residential buildings according to the ISO 10137:2009*

By analysing the parameters in Equation (2) and in Table 6, the main reasons for the EKS method yielding the highest peak accelerations are higher wind pressure q_{zref} and higher resonance response factor R. In the EKS model, the wind pressure at the building height h is used whereas the wind pressure at the reference height 0.6 h is used in the other codes.

Although the global peak factor estimated by the EKS are lower than the ASCE. The turbulence intensity I_v and the peak gust factor k_p are indeed larger in the American code. The global peak factor in the EKS uses a factor 2 whereas the American one uses 1.7 (see Table 1).

But the main difference in acceleration predictions comes from the resonance response factor R which is much higher for the EKS. According to the equations for R^2 in Table 3 and the evaluated parameters in Table 7, this is due to the size factor S that is about four to six times higher in the EKS method compared to the ASCE. The reasons are much higher wind pressure correlation factors on the surfaces normal to the wind, Φ_h and Φ_b, in the EKS. In Figure 4, the correlation size reduction R_h/Φ_h and R_b/Φ_b as functions of height/width are plotted for the different codes. The continuous curves correspond to cases where $n_{1,x}/v_{m,zref}$ is equal to 0.02 m^{-1} and the dashed curves where $n_{1,x}/v_{m,zref}$ is equal to 0.06 m^{-1}. For the case of the building TREET, the vertical size reduction factor according to the ASCE method is R_h = 0.086 and according to the EKS method it is Φ_h = 0.210. The horizontal correlation along-wind is not considered in the EKS, whereas in the ASCE it is included in the size factor S with the term $(0.53+0.47R_d)$. The vertical and the horizontal wind pressure correlation functions, R_h and R_b, are identical functions with respect to h and b and are the same in the ASCE and in the EC-B. In the EKS the functions for horizontal and vertical correlation are different. The yellow curves correspond to vertical correlation Φ_h according to the EKS and is much higher than the size correlation R_h = R_b according to the ASCE/EC-B for building height between 50 and 100 m.

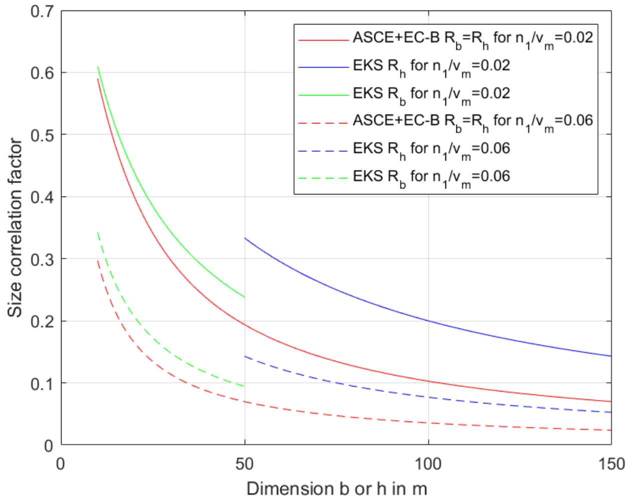


Figure 4: *Wind pressure correlation factors R_h/Φ_h and R_b/Φ_b as a function of the building height h and width b in meter for ranges of $n_{1,x}/v_{m,zref}$ between 0.02 and 0.06 m^{-1}*

The energy factors E estimated with the American code are slightly higher and the total damping values ξ_{tot} in the Swedish method, including aerodynamical damping, are higher. But the high correlation factor S of the EKS, as mention before, is strongly influencing the resonance response factor R and therefore explain the higher peak accelerations.
For the studied building cases the non-dimensional frequencies ranged between 3 and 8. Taller buildings with lower non-dimensional frequencies might results in higher energy factors in the case of EKS, as illustrated in Figure 5 which represents the wind power spectral densities as functions of the non-dimensional frequency for the different codes.

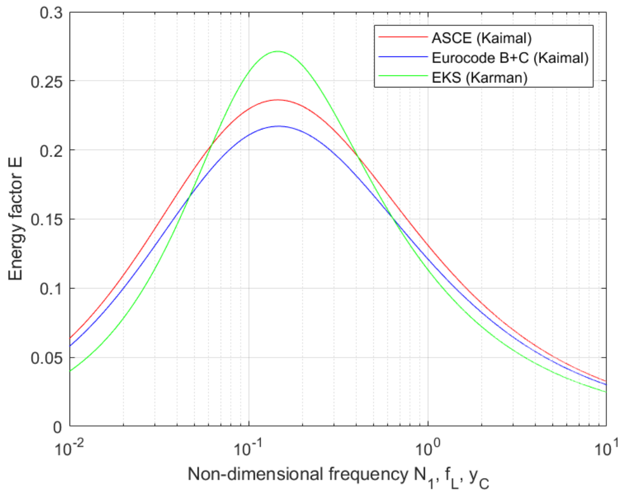


Figure 5: *Wind power spectral densities as functions of the non-dimensional frequency N_1, f_L or y_C*

In the specific case of SARA Cultural Centre and Mjøstårnet with the highest peak acceleration difference between ASCE and EKS, the ratios K/M_1 differ about 20 % but the ratio K/M_1 is almost the same for the other buildings. The main reason is that the EKS does not consider the non-uniform distribution of the mass vertically and evaluate the modal mass from the mean building density. Increased mass at the top floors of tall buildings is one of the most common way to mitigate the serviceability related to wind-induced vibrations. Traditional tall buildings about 180-200 m tall made of steel and reinforced concrete have building densities between 175 and 250 kg/m^3 [1,8,19].
When comparing the acceleration results from the Eurocode methods EC-B and EC-C the differences are due to various correlation size factors S.
The differences between the acceleration results from the EC-B and the EKS methods are due to the same differences described previously for the comparison between the ASCE and the EKS methods.
The influence of the mode shape exponent on the peak acceleration seems low. The EKS is derived with a mode shape exponent fixed at 1.5 whereas the other methods leave this variable to the structural designer, usually between 1 and 2. The assumed mode shape of a building reflects the vertical distribution of bending or/and shear motion. Several wind aerodynamic parameters are evaluated at 100 % of the building height in the EKS: the aerodynamic damping, the turbulence intensity and indirectly the energy factor E. For the ASCE and the Eurocodes they are evaluated at 60 % of h.

5 CONCLUSIONS

The phenomenon of wind-induced vibration is an important criterion for design of tall timber buildings already when their height reaches levels of 15-20 storeys, due to the combination of relatively low stiffness and low mass. For tall buildings in traditional material this phenomenon does not come into importance until they reach much higher levels. This makes it necessary to revisit the calculations of acceleration according to wind codes for light-weight buildings. To provide good comfort at the top floor, the building acceleration must not exceed some threshold value. Wind loads are hard to model due to complex turbulent space-time variation, and building codes provide different methods based on various assumptions and empirical data.
The comparison of four procedures from Europe and America yields factor ratios over 3.0 for the along-wind accelerations evaluated for five tall timber buildings which heights are between 40 and 100 m. The vertical correlation factor, the reference height for the mean wind pressure and the mass distribution are not estimated in the same way in the four building codes investigated. They lead to huge differences in the predicted peak accelerations which can influence the design for wind-induced vibrations. The analysis of the comparative study emphasizes a considerable uncertainty in the building codes for the response of taller and lighter timber buildings excited by the wind.

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