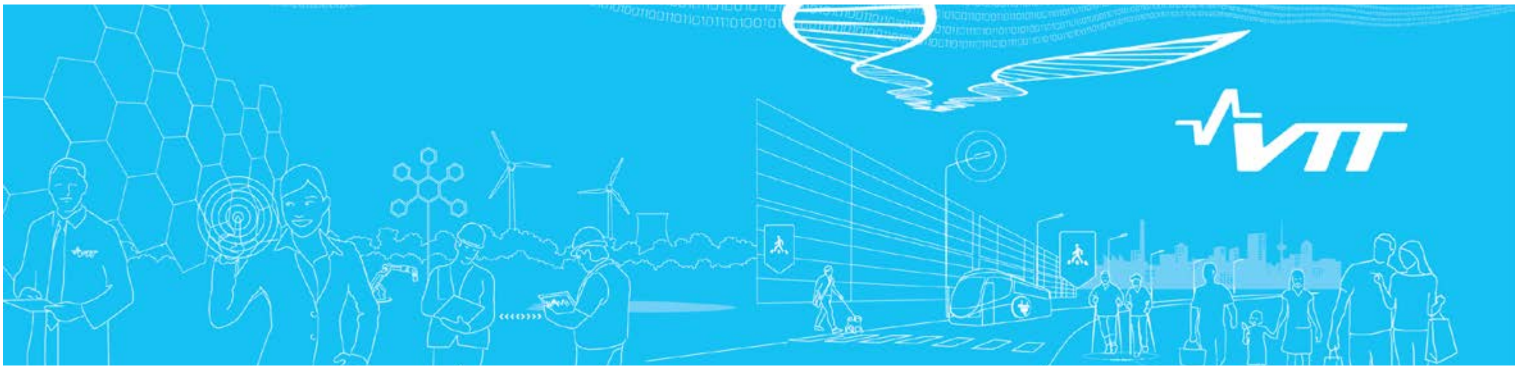

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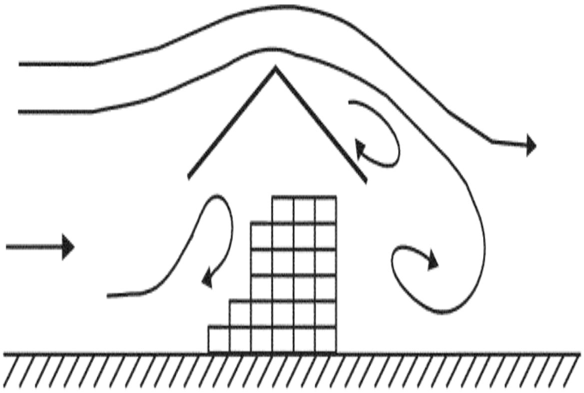

Source: EN1991-1-4: General actions. Wind actions

Evaluation of wind-induced vibrations of modular buildings

Authors: Asko Talja, Ludovic Fülöp

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Evaluation of wind-induced vibrations of modular buildings	
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Summary

The report systematically presents the Eurocode standard method for wind vibration calculations. The different interpretations, assumptions and limitations of the standard are discussed. Based on a large number of model calculations for building typologies, representative for possible CLT modular configurations, a simplified evaluation formula is suggested. The formulation, takes into account the main physical parameters affecting wind-direction vibrations, the wind velocity, the unit height mass, the natural frequency and the damping of the structures, and is compared with EN1991-1-4, Annex C calculations and earlier similar proposals from the international literature by Carpenter et al (2013).

For defining possible vibration limits, several international reference documents are reviewed (ISO 10137, AIJ 2004). It is emphasised that human perception and tolerance to wind-induced building vibration are subjective matter. Therefore, limits satisfying all occupant expectations cannot be found. Instead it is sensible to use a probabilistic representation of the limit ranges (Figure 21), where the likelihood of customer satisfaction is correlated with the limit criteria for vibrations. Such probabilistic limits help clarify that, whatever the vibration level there is always a residual probability of discomfort by a certain percentage of occupants.

However, admitting that the use of probabilistic limits may be less practical in the commercial domain, we suggest the use of SFS-EN ISO 8041. 2005 with some adjustments for imposing acceptable limits on wind induced vibrations.

Espoo, 22.9.2016

Written by	**Reviewed by**	**Accepted by**
Asko Talja, Senior Scientist Ludovic Fülöp, Principal Scientist	Vilho Jussila, Research Scientist	Mikko Lehtonen, Research Team Leader

VTT's contact address

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Contents

1. Description and objectives

In cross laminated timber (CLT) buildings insulation of structure-borne noise of buildings is often based on the use of elastic insulation strips between CLT panels. The presence of elastic strips between the CLT panels and the lower elastic modulus of timber may results in increased flexibility the buildings, even if the basis shear wall building configuration results in higher stiffness than framed structures. Hence, sensitivity to wind-induced horizontal vibrations may be perceived as a design issue for CLT structures. Vibrations reduce the comfort of living, and therefore design guidance is needed in order to avoid possible resident complaints.

This study describes the background of evaluation of wind-induced vibrations of buildings. It covers determination of wind loads used in vibration design, methods for calculating vibration levels of modular buildings and determination of acceptance levels of vibrations. The findings will be later used as basis for design guidance.

A very general picture of how wind induced vibrations is taken into account in buildings is presented in Figure 1. The starting point is a spectral representation of the wind velocity (S_{vv}), which is a frequency (n) dependent velocity model for the wind speed (Figure 1). The load spectrum (S_{FF}) is calculated using the admittance function (H_a), depending on the exposed area and surface properties of the wind-exposed building elements, (Figure 1). S_{FF} is a frequency dependent model for the wind induced loads expressed in pressures or forces acting on the construction elements. Once forces are known, the next evaluation step is to estimate building responses (S_{uu}) by using the transfer function reflecting the mechanical properties of the structure/element (H_s). Obviously, the response is dependent on the vibration frequencies of the structure/element (n_i), and primarily on the natural frequency. Hence, the response (S_{uu}) will be amplified at the vibration frequencies of the elements. The final step of the assessment, not represented in Figure 1, is to compare the resulting vibration responses (S_{uu}) with acceptance criteria, developed based on human sensitivity to vibrations and expectations concerning occupant comfort.

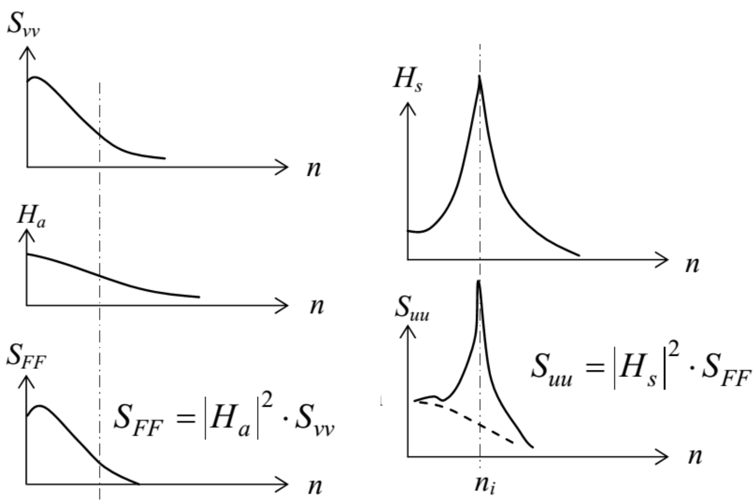


Figure 1. General framework of wind inducing vibrations in buildings (Steenberger et al. 2009).

The different steps of the above generic calculation framework are described below.

2. Determination of wind velocities used in vibration design (magnitude and frequency spectrum)

Wind load is a fluctuating load. The wind velocity is characterised by a mean value, and a fluctuating part around this mean. The fluctuating part of the wind load causes the dynamic behaviour of the buildings. The EN1991-1-4 (2005) standard can be used to estimate wind velocities, wind loads and vibration effects for buildings and civil engineering works up to a height of h ≤ 200 m, and bridges having a span not greater than 200 m.

2.1 General consideration and limitations of EN 1991-1-4

In the design load combinations, wind action should be considered as variable fixed load (Clause 3.3, 1991-1-4). Other loads (e.g. live load, snow, ice etc.) which modify the wind effect should be taken into account together in combination with the wind action. These actions may act by changing the dynamic properties of the building or changing the exposed area to wind, or both.

It is especially important to consider the live load (floor load) acting together with wind load. As it will be shown later, the floor mass has an effect on the design, and assuming large floor-masses leads to un-conservative estimates.

The wind actions calculated according to EN 1991-1-4 (2005) are characteristic values with a return period of T_R = 50 years, or 2% probability of being exceeded a year. However, for calculations concerning vibration comfort it is more usual to use return period of T_R = 1 year, because acceptance criteria is often given for 1 year period (e.g. ISO 10137, Figure 18).

In the design, the building's dynamic response is calculated due to along wind turbulence in resonance with the along-wind fundamental mode of vibration (Clause 3.5, EN 1991-1-4). Hence, this is the kind of vibration assessed in a basic design case.

2.2 Wind velocities in EN 1991-1-4

Wind action can be represented as wind velocity, described by a mean and a fluctuating component (Figure 2). The mean component is usually used for calculating static deformations, while the fluctuating component is used to estimate vibrations.

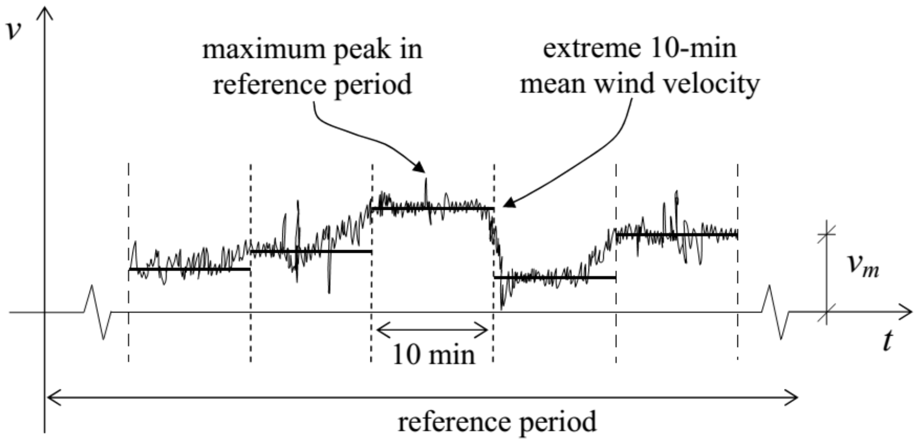


Figure 2. Reference period is 10 minutes (T = 600 s) in EN 1991-1-4 for the calculation of the mean and fluctuating components of wind (Steenberger et al. 2009).

The primary representation of wind is by velocity. The wind velocity definition starts in EN 1991-1-4 from the basic wind velocity:

$$v_b = c_{dir} \cdot c_{season} \cdot v_{b,0} \qquad 1$$

EN1991-1-4, Eq. 4.1

Where:

- v_b is the basic wind velocity 10 m above the ground;
- c_{dir} and c_{season} are factors to account for directional and seasonal changes in the wind velocity, and
- $v_{b,0}$ is the fundamental value of basic wind velocity. $V_{b,0}$ is given in the Finnish National Annex (NA) of EN 1991-1-4 to be between 21 m/s to 26 m/s, depending on the geographic region in the country (i.e. mainland, Lapland lowland areas, sea areas, Lapland mountain-top areas). These are values with T_R = 50 years return period. It is also worth noting that a draft of the Finnish Ministry of the Environment, is foreseeing prescribing 21 m/s for the whole territory (Ympäristö, 2016).

As mentioned earlier, for vibration comfort calculations, a return period of T_R = 1 year wind speed (v_{b1}) is needed. This can be obtained by multiplying the 50 years v_b value with a reduction factor c_{prob}. For any annual exceedance probability p:

$$c_{prob} = \left(\frac{1 - K \cdot \ln(-\ln(1-p))}{1 - K \cdot \ln(-\ln(0{,}98))} \right)^n \qquad 2$$

EN1991-1-4, Eq. 4.2

Where:

- K is a factor depending on the coefficient of variation (COV) of the extreme-value distribution for wind (K = 0.2);
- n exponent (n = 0.5);
- p is the desired annual exceedance probability. The return period and annual exceedance probability are linked by the relationships: $T_R = -1/\ln(1-p)$ and $p = 1-\exp(-1/T_R)$. Hence, for a return period of T_R = 1 year the values are p = 0.63, c_{prob} = 0.75 and $v_{b1} = v_b * c_{prob}$.

The basic wind velocity (v_b) is used to define the mean wind velocity $v_m(z)$, which depends on the terrain type around the building and the height at which velocity is calculated (z):

$$V_m(z) = c_r(z) \cdot c_o(z) \cdot V_{b1} \qquad 3$$

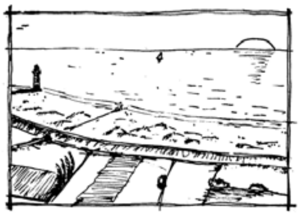
EN1991-1-4, Eq. 4.3

Where:

- $v_m(z)$ is mean wind speed at height z;
- $c_r(z)$ is the terrain roughness factor;
- $c_O(z)$ is the orography factor.

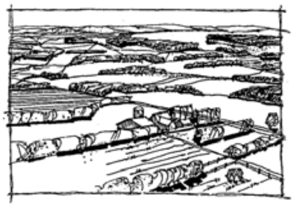
Within the same geographic region there can be large variations of wind velocity depending on the type of landscape immediately surrounding the building. This variation is taken into account by the factor $c_r(z)$ and $c_O(z)$ in the above relationship. The roughness factor depends on the terrain category. EN 1991-1-4 gives terrain categories classified depending on the type and number of obstacles (Figure 3).

Terrain category 0
Sea, coastal area exposed to the open sea



Terrain category I
Lakes or area with negligible vegetation and without obstacles

Terrain category II
Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights



Terrain category III
Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)

Terrain category IV
Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m

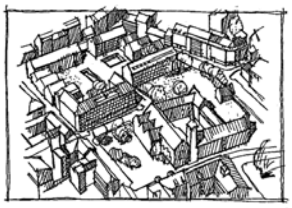


Figure 3. The meaning of terrain category 0, I, II, III and IV as illustrated in EN1991-1-4.

The terrain category to be applied depends on expected wind direction. The code recommends including in the estimation the terrain within 30 deg's of angular range and distance depending on the height of the building (Table A1, EN1991-1-4). In case of higher building (h = 30–50 m) the distance can increase considerably, up to 20–50 km in the down wind direction (Figure 4). Hence, the designer has to examine quite large area around the building. It is also recommended to consider the prevailing terrain and ignore patches occupying less than 10% of the area.

It is hence likely, that several $c_r(z)$'s can be defined for a building depending on the attack direction of the wind. If this effect and the shape of the building in plan (e.g. long rectangular shapes) are interacting, several wind load scenarios should be considered for a single design.

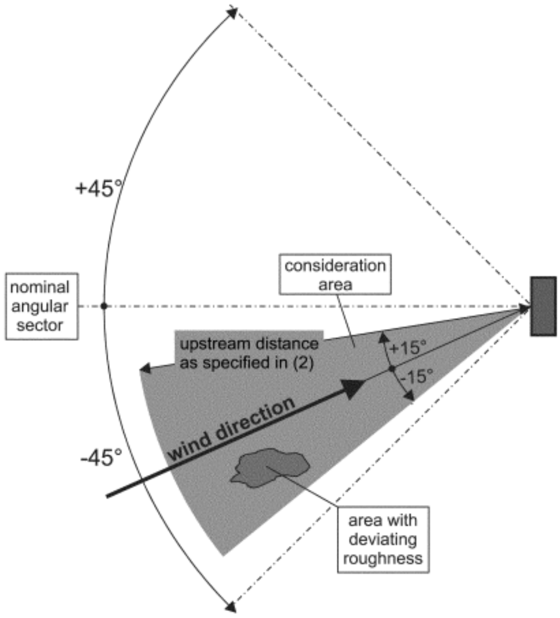


Figure 4. Area to be considered for classifying terrain type (EN1991-1-4).

The terrain orography (i.e. topography of hills/mountains) factor takes into account local exposure or sheltering of the building. If the building is on crest of a hill, this particular position will have an increased exposure even within the terrain category surrounding it. On the contrary, a location on a down-wind slope of a hill provides sheltering within the terrain category.

$c_o = 1$	for	$\Phi < 0{,}05$	4
$c_o = 1 + 2 \cdot s \cdot \Phi$	for	$0{,}05 < \Phi < 0{,}3$	EN1991-1-4, Eq. A.1, A.2 and A.3
$c_o = 1 + 0{,}6 \cdot s$	for	$\Phi > 0{,}3$	

Where:

- $c_O(z)$ is the orography factor at height z;
- Φ is the upstream slope $\Phi = H/L_u$ (Figure 5); and
- s is a location factor, depending on the position of the building within the slope/hill scenario (Figures A.2 and A.3 in EN 1991-1-4). The maximum value of *s* given in EN1991-1-4 is s = 0.8. With the most disadvantageous slope this results in max value of the $c_O(z)$, so $c_O(z)$ = 1–1.5.

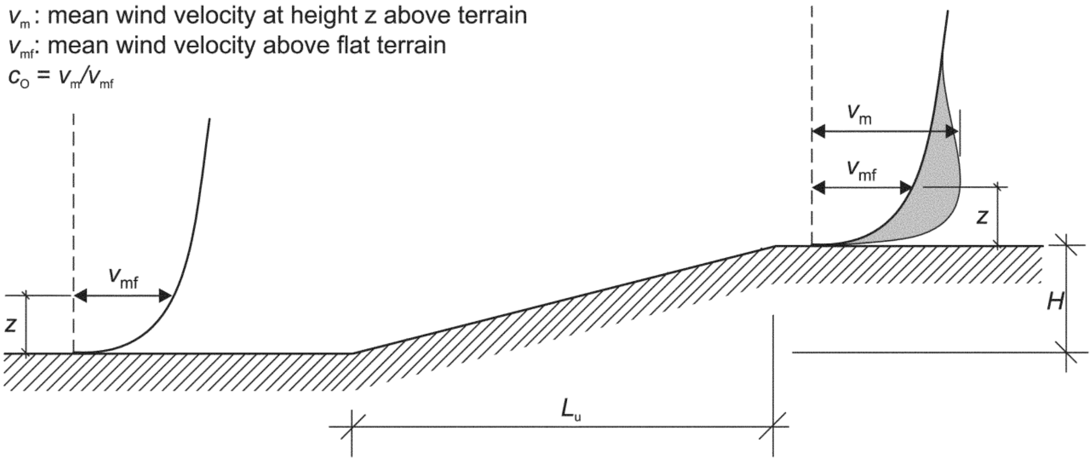


Figure 5. Definition of c_O (EN1991-1-4).

The fluctuating/turbulent component of the wind (see Figure 2), is given by the turbulence intensity $I_v(z)$ at any calculation height z:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_o(z) \cdot \ln(z/z_0)} \quad \text{for} \quad z_{min} \leq z \leq z_{max}$$
$$I_v(z) = I_v(z_{min}) \quad \text{for} \quad z < z_{min}$$

5

EN1991-1-4, Eq. 4.3

Where:

- $v_m(z)$ is mean wind speed at height z, and
- σ_v is the standard deviation of the turbulent component.
- z_{max} is the maximum height where EN1991-1-4 should be used (z_{max} = 200 m);
- z_{min} is the minimum height limit for each terrain category (z_{min} = 1–10 m, Table 4.1, EN1991-1-4).

The first part of the equation clarifies the statistical meaning of $I_v(z)$, as the ratio between the standard deviation and the mean velocity. The second part of the equation gives the means to calculate $I_v(z)$, using:

- k_I a turbulence factor;
- $c_O(z)$ the orography factor and
- z_0 the roughness length given for each terrain category.

Hence, using Equations 1 and 5, a complete stochastic description is obtained for the wind velocity. However, the effect of the wind on the construction elements is only indirectly described by velocity, the effects sought by engineers are pressures and forces.

3. Methods for calculating vibration levels of modular buildings

Because of the dynamic behaviour of the building, the maximum load effect can be larger compared to the situation of only static response. Therefore, in the building design, a dynamic amplification factor has to be taken into account.

For estimating vibrations it is not needed to calculate the mean loads (pressures or forces) induced by the wind. Hence, the following section describes only briefly the EN1991-1-4 calculation of loads.

3.1 Wind pressures or forces

The wind load on the building elements can be considered:

- to be acting in a distributed way, perpendicular to the surface (pressure) or tangential to the surface (friction);
- as wind force for the whole structure.

From the point of view of the vibration of the entire building, it is the total force acting on the whole structure which is important. This can be calculated by estimating forces using force coefficients (c_f), or calculating forces from surface pressures, using the external pressure (w_e), internal pressures (w_i) and friction coefficient (c_{fr}). The direct force estimation is carried with the expression:

$$F_w = c_s c_d \cdot \sum_{\text{elements}} c_f \cdot q_p(z_e) \cdot A_{ref} \qquad (6)$$

EN1991-1-4, Eq. 5.4

Where:

- c_s and c_d are the structural factors (Section 6, EN 1991-1-4)
- c_f is the force coefficient (Section 7, EN1991-1-4)
- $q_p(z_e)$ is the peak velocity pressure at the reference height z_e, which include mean and short-term velocity fluctuations,
- A_{ref} is the reference area of the structural elements

Since, the surface of the building is usually large and composed of complex shapes, individual forces are calculated for elements or regions. Each element or region has area A_{ref} and the resulting forces for each element can be summed as vectors (Σ).

3.2 Wind induced vibrations

EN 1994-1-1 has two informative (non-compulsory) annexes which include methods for the evaluation of displacements and accelerations for serviceability assessment (Annex B and Annex C). Comparisons with the theoretical solution show that the Annex B method is providing un-conservative estimates by about 40% (Steenberger et al. 2009). Hence, in the following section only the method of Annex C is summarised, with special focus on horizontal floor accelerations, as main characteristic affecting occupant comfort.

The estimation of acceleration can be made in any location (y, z) and is based on the natural vibration frequency in the down-wind direction ($n_{1,x}$) and associated mode shape (Φ_i)

presented in Figure 6.a. The height dependent quantities (e.g. mean wind velocity $v_m(z)$) are estimated to a conventional height of z_s = 0.6*h for rectangular buildings (Figure 6.b). For other building configurations different conventional height may be used, with the upper limit of z_s = h for a conservative estimate.

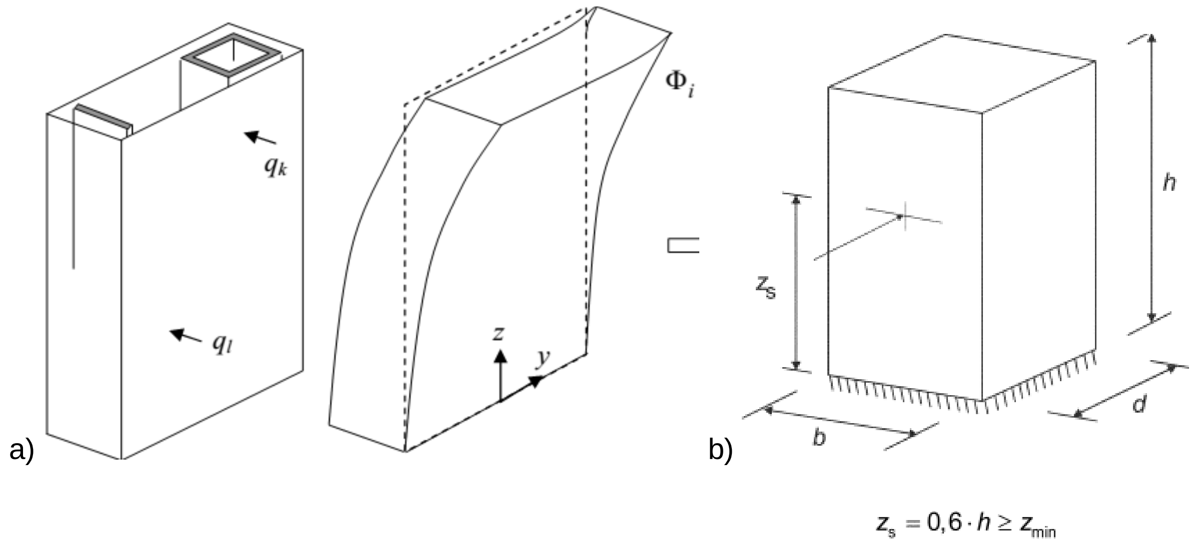


Figure 6. Vibration behaviour of buildings with pressures (q_k, q_l) natural mode (Φ_i) emphasised (Steenberger et al. 2009) and (b) definitions of main dimensions in EN 1991-1-4 (2005).

First, the turbulent length scale L(z) is calculated for the height z_s:

$$L(z) = L_t \cdot \left(\frac{z}{z_t}\right)^{\alpha} \quad \text{for} \quad z \geq z_{min}$$

$$L(z) = L(z_{min}) \quad \text{for} \quad z < z_{min}$$

7

EN1991-1-4, Eq. B.1

Where:

- $z = z_s$ is the height where the value is calculated, z_t = 200 m is the reference height and z_{min} is given in Table 4.1 of EN1991-1-4.
- L_t is a reference length scale with L_t = 300 m.
- $\alpha = 0,67 + 0,05 \ln(z_0)$, with z_0 is the roughness length in meters (Table 4.1 of EN1991-1-4). This value varies between 0.003 m and 1 m.

The frequency distribution of wind velocity is expressed as power spectral density function $S_L(z, n)$. The graphical representation of $S_L(z, n)$ is given in Figure 7.

$$S_L(z,n) = \frac{n \cdot S_v(z,n)}{\sigma_v^2} = \frac{6,8 \cdot f_L(z,n)}{(1 + 10,2 \cdot f_L(z,n))^{5/3}}$$

8

EN1991-1-4, Eq B.2

Where:

- f_L is a non-dimensional frequency, calculated from the fundamental frequency in the vibration direction ($n = n_{1,x}$) by the formula: $f_L(z,n) = \frac{n \cdot L(z)}{v_m(z)}$, with the mean wind speed $v_m(z)$ also playing a role. It has to be noted that f_L, unlike $n_{1.x}$ is not solely the property of the building, but it is also affected by the wind conditions

- $z = z_s$ is the height where the value is calculated.

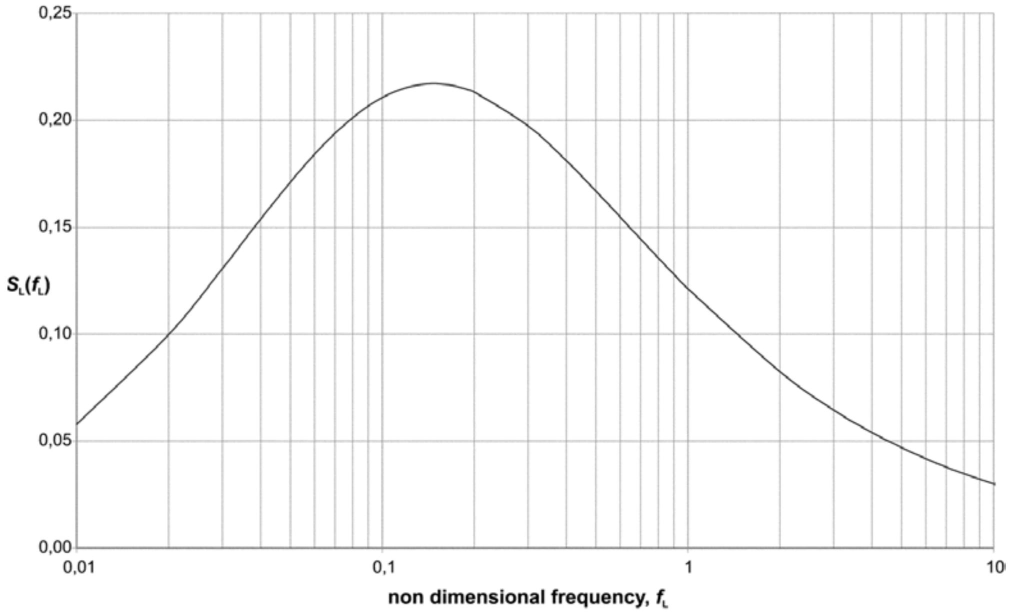


Figure 7. Power spectral density function $S_L(n,z) = S_L(f_L(n,z)) = S_L(f_L)$.

In the next step, the resonance response factor R^2 should be calculated. This factor takes into account the amplification due to possible resonance between the fluctuating component of the wind the and the fundamental vibration mode of the building:

$$R^2 = \frac{\pi^2}{2\cdot\delta}\cdot S_L(z_s, n_{1,x})\cdot K_s(n_{1,x})$$

9

EN1991-1-4, Eq. C.2

Where:

- δ is the logarithmic decrement of damping (Annex F of EN1994-1-4);
- $S_L(z_s, n_{1.x})$ is the spectral density function given earlier;
- $n_{1,x}$ the natural frequency of the structure;
- K_s the size reduction function, depending on the assumed mode shape:

$$K_s(n) = \frac{1}{1+\sqrt{(G_y\cdot\phi_y)^2 + (G_z\cdot\phi_z)^2 + \left(\frac{2}{\pi}\cdot G_y\cdot\phi_y\cdot G_z\cdot\phi_z\right)^2}}$$

10

EN1991-1-4, Eq. C.3

$$\phi_y = \frac{c_y\cdot b\cdot n}{v_m(z_s)} \qquad \phi_z = \frac{c_z\cdot h\cdot n}{v_m(z_s)}$$

Where:

- G depends on the mode shape variation in the *y* and *z* directions (Figure 6) and they are usually G_y = 1/2 (uniform function) and G_z = 3/8 (linear function) or = 5/18 (parabolic function). A more detailed discussion of mode shape approximations in the z direction is given with Figure 8.

- c_y and c_z are both 11.5;
- $n = n_{1,x}$ the natural frequency of the structure;
- h and b are the height and width of the building.

It is worth mentioning here that the damping δ is the sum of the logarithmic decrement of the structural damping (δ_s), of the aerodynamic damping (δ_a) and of the damping coming from specialized damping devices (δ_d).

The values for logarithmic decrement of the structural damping (δ_s) are given in Table F.5, EN1991-1-4 as $\delta_s = 0.1$ for concrete and $\delta_s = 0.05$ for steel structures. These values can be converted to damping ratios from the relationship: δ = 2*π*ξ/sqrt(1-ξ^2), or because sqrt(1-ξ^2)~1 from the simplified expression δ~2*π*ξ. So, logarithmic decrement δ = 0.1 and 0.05 is equivalent to damping ratio ξ = 0.016 and 0.08 respectively (i.e. 1.6% and 0.8%).

The aerodynamic damping can be calculated as:

$$\delta_a = \frac{c_f \cdot \rho \cdot v_m(z_s)}{2 \cdot n_1 \cdot \mu_e}$$ 11

EN1991-1-4, Eq. F.16

Where:

- c_f is the same force coefficient (Section 7, EN1991-1-4) as used in Equation 6. Specifically Equation 7.9 of EN1991-1-4 can be used for rectangular buildings;
- ρ is the air density (= 1.25 kg/m^3), Clause 4.5, EN1991-1-4;
- $v_m(z_s)$ is mean wind speed at height $z = z_s$;
- $n_1 = n_{1,x}$ the natural frequency of the structure, and
- μ_e is the equivalent mass which can be approximated by the mass per unit area at the top of the building. <u>*Note on masses*</u>: This is mass/unit façade area $\mu_e = M_{TOT}/(h*b)$ in kg/m^2, when the building is modelled as a continuum plate (see Equation F.16 & F17, EN1991-1-4). When the model is a cantilever beam, the mass per unit height (m_e) is used in EN1991-1-4, $m_e = M_{TOT}/h$ (kg/m). In the literature the mass density is also often used, $M_{TOT}/(h*b*d)$ in kg/m^3. M_{TOT} is the total mass of the building considered for the wind load situation, including self-weight and a fraction of the live load. Lower mass results in higher vibrations, so increasing live load makes the results less conservative.

The usual target to be calculated from wind induced vibration is acceleration. The characteristic peak acceleration is obtained by multiplying the standard deviation of the acceleration ($\sigma_{a.x}$) by the peak factor (k_p):

$$A_x(y,z) = \sigma_{a.x}(y,z) \cdot k_p$$ 12

The standard deviation ($\sigma_{a.x}$) of the wind direction acceleration of the structural point y, z (Figure 6) is given by:

$$\sigma_{a,x}(y,z) = c_f \cdot \rho \cdot I_v(z_s) \cdot v_m^2(z_s) \cdot R \cdot \frac{K_y \cdot K_z \cdot \Phi(y,z)}{\mu_{ref} \cdot \Phi_{max}}$$ 13

EN1991-1-4, Eq C.4

Where:

- c_f is the force coefficient (determined for the entire building, or calculated as total effect of pressures, suction and friction on the surfaces of the building);
- ρ is the air density (= 1.25 kg/m³), Clause 4.5, EN1991-1-4;
- $I_v(z_s)$ is the turbulence intensity at height z_s (Figure 6);
- $v_m(z_s)$ is the characteristic mean wind velocity at height z_s (Figure 6);
- R is the square root of the resonance factor (Equation 9);
- K_y and K_z are constants with default values of K_y = 1 and K_z = 3/2 or = 5/3 for rectangular buildings (Table C.1, EN1991-1-4);
- $\mu_{ref} = \mu_e$ is the reference mass per unit area (F.5(3), EN1991-1-4;
- Φ_{max} is the mode shape value at maximum amplitude (= 1 is mode shape is normalised to top displacement Φ_{max} = 1);
- Φ(y,z) is the mode shape value at the calculation coordinate *y*, *z*. If the mode is of pure bending, like in symmetrical buildings, than only the height *z* plays a role, because the mode shape is constant with *y*. Over the height, the variation can be approximated as uniform, linear or parabolic.

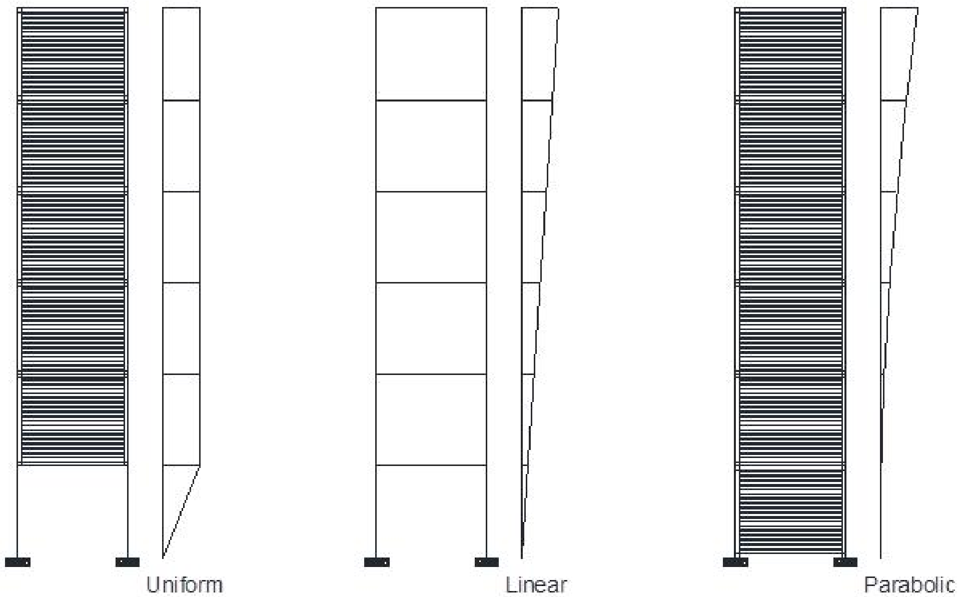


Figure 8. Approximate mode shapes for typical buildings.

The peak factor (k_p) is obtained by:

$k_p = \sqrt{2 \cdot \ln(\nu \cdot T)} + 0.6/\sqrt{2 \cdot \ln(\nu \cdot T)}$; k_p>=3 14

EN1991-1-4, Eq B.4

Where:

- *v* the up-crossing frequency (see below);

- T is the averaging time of the mean wind velocity (T = 10 min = 600 s, in EN1991-1-4);

$$\nu = n_{1,x}\sqrt{\frac{R^2}{B^2 + R^2}} \quad ; \quad \nu \geq 0{,}08\ Hz$$

15

EN1991-1-4, Eq B.4

Where:

- $n_{1,x}$ is the natural frequency of the structure;
- R^2 is the resonance response factor given in Equation 9, and
- B^2 is the background factor allowing for the lack of correlation of peak-pressure on the buildings surfaces from turbulent wind. The background factor can be calculated using the formula below. Choosing B^2 = 1 is conservative:

$$B^2 = \frac{1}{1 + \frac{3}{2}\cdot\sqrt{\left(\frac{b}{L(z_s)}\right)^2 + \left(\frac{h}{L(z_s)}\right)^2 + \left(\frac{b}{L(z_s)}\cdot\frac{h}{L(z_s)}\right)^2}}$$

16

EN1991-1-4, Eq C.1

Where:

- The values of b, h and z_s are given in Figure 6.
- L(z) is the turbulent length scale (see Equation 7) at the height z_s.

3.3 Discussion and factors to be considered for CLT buildings

A set of calculation examples was presented by Steenberger et al. (2009) for generic buildings, in a parametric fashion. They used a theoretical method and Procedure 1 and Procedure 2 (Annex C) to estimate wind-direction vibrations. The Proc. 2 results are based on Annex C, EN1991-1-4 calculations.

They found that Proc. 2 (Annex C) agrees with the theoretical estimates. The mode shapes seem to have important impact on the results, which is not a surprise. Most likely the fundamental mode of a 100 m+ building is better represented by a parabola than a linear function. For these high buildings, the effect of terrain category (z_0) is not very significant.

Table 1. Acceleration estimates for h = 100, 150, 200 m buildings, v_b = 15 m/s and fundamental frequency n = 0.2, 0.3, 0.4 Hz, damping ξ = 1%.

height	n	v_b	z_0	mode	$\hat{a}$, theory	$\hat{a}$, proc1	$\hat{a}$, proc2	% error proc 1
200	0.2	15	0.1	linear	0.038	0.032	0.039	24 %
				parabolic	0.049	0.036	0.050	39 %
			0.01	linear	0.040	0.034	0.042	23 %
				parabolic	0.052	0.039	0.053	38 %
150	0.3	15	0.1	linear	0.027	0.022	0.028	24 %
				parabolic	0.035	0.025	0.036	41 %
			0.01	linear	0.029	0.025	0.031	24 %
				parabolic	0.038	0.028	0.039	41 %
100	0.4	15	0.1	linear	0.022	0.018	0.023	24 %
				parabolic	0.029	0.021	0.029	40 %
			0.01	linear	0.025	0.021	0.026	24 %
				parabolic	0.032	0.023	0.033	40 %

Even if Steenberger et al. (2009) presents generic calculations, without even defining the building materials, one can tentatively compare the results in Table 1 with the requirements in Figure 21. Such comparison indicates that about 30–50% of the occupants would perceive disturbance from along-wind vibrations for the configurations chosen (Figure 9). It is also interesting to note that the percentages do not necessary improve with decreasing height of the building.

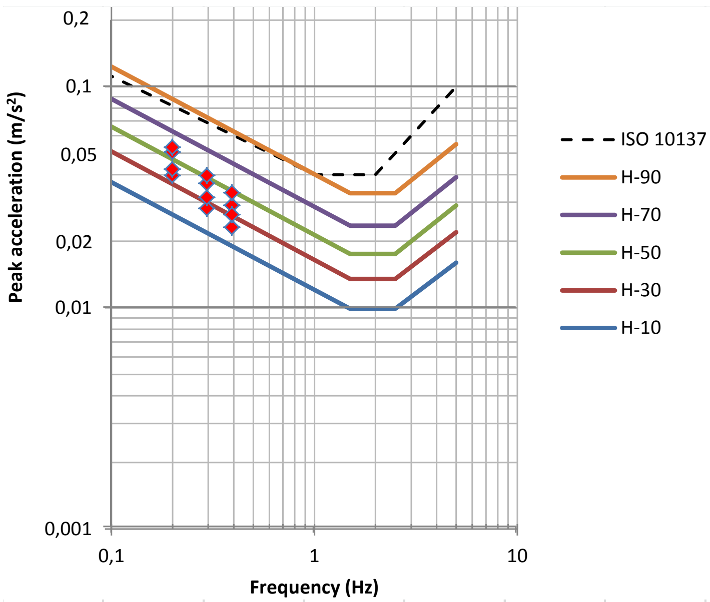


Figure 9. Comparison of wind velocities (red diamonds) from Steenberger et al (2009), with the AIJ probabilistic limit curves (Figure 21).

Some considerations result for CLT building:

- The vibration shape has to be considered carefully. This will be available from a FEM building model and hence proper approximation need to be used.

- The natural frequency ($n_{1.x}$) of the building is important. Some reference values can be collected from the literature (e.g. n_1 = 1.69 Hz for h = 24 m, Kryh and M. Nilsson 2012). It is most likely that the FEM model used in the design will overestimate the stiffness of the building. Empirical equations also exist for predicting natural period as a function of height. Studies show that the fit of empirical equations with measurements is often better than the fit of FEM estimates with measurement. Unfortunately, no specialised CLT building empirical equation exist to predict natural frequency, while some have been developed for steel and reinforced concrete buildings.

  Eurocodes allow the approximation of the natural frequency as:

$$n_1 = \frac{46}{h} \tag{17}$$

Where h is the total height of the building in meters and n_1 results in Hz.

A more refined equation for first mode of vibration takes into account the depth (d) and the height (h) of the building. It was proposed by ECCS (1987), and it is:

$$n_1 = \frac{\sqrt{d}}{0.1 \cdot h} \tag{18}$$

Where the height (h) and depth (d) are in meters and n_1 results in Hz.

Similar proposals were made by Tamura, differentiating between building types. For steel structures the height factor was 0.02 (Figure 10.a), equivalent of using 50 in equation 17, and 0.015 (Figure 10.b) for concrete structures (equivalent of 66.7 in equation 17).

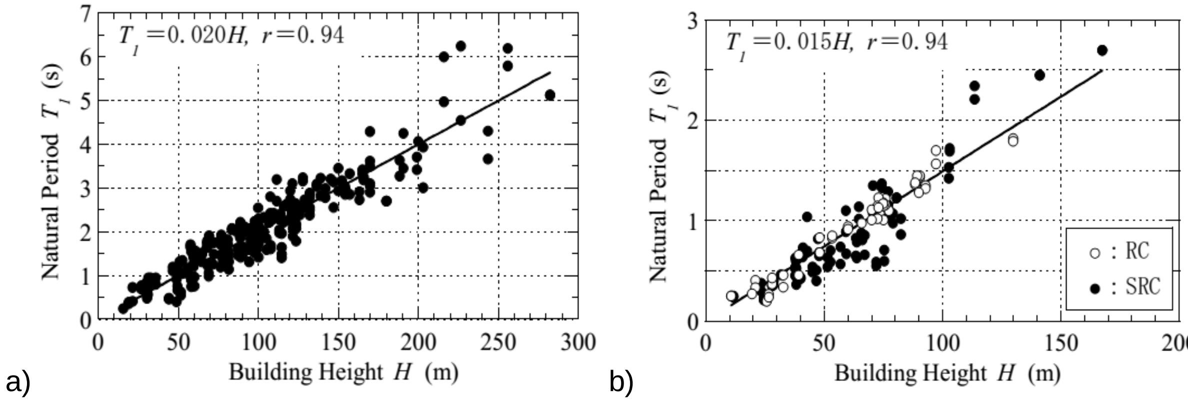


Figure 10. Fit of empirical equation and measurements predicting the natural frequency of (a) steel and (b) concrete buildings (Tamura, NA). In this figure H=h is the height of the building.

- Another possible factor to consider is damping. Damping plays a role in defining R (Eq. 9) directly influencing the standard deviation of the along-wind acceleration (Eq. 13). As mentioned, the damping is the sum of structural damping (δ_s), aerodynamic damping (δ_a) and damping from specialized devices (δ_d). No values exist for structural damping of CLT buildings in the design code EN1991-1-4.

- Finally, it can be that other than along-wind vibration is playing a role in typical CLT buildings, e.g. if stiff core construction is used, or floors are not stiff enough to assure a uniform mode of vibration. Measurements for typical steel structures suggest that the building are torsionaly stiffer with a ratio of torsional mode to natural mode of about 0.75 (Figure 11). This may or may not be the case for typical CLT buildings.

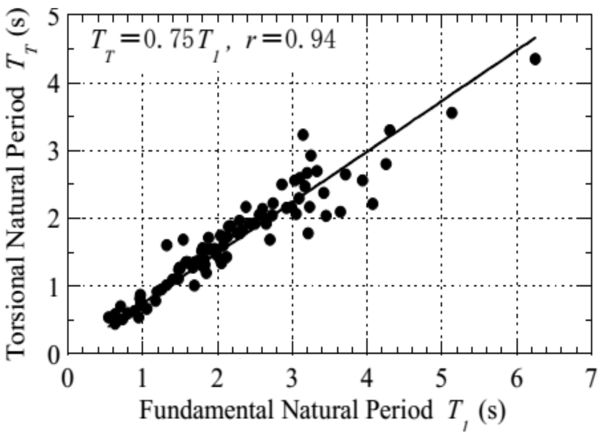


Figure 11. Fit of torsional mode to natural frequency of steel buildings (Tamura, NA).

4. Simplified wind-direction vibration calculations in CLT buildings

The above methodology was applied for calculating the wind induced vibrations of a family of CLT buildings with practical configurations. The results of these calculations show that simplifications can be made to the methodology, as long as the following assumptions are respected:

- Rectangular buildings with height (h) between 10 and 140m and width (b) in the range of 10–60 m for low buildings and 30–35 for high buildings. The depth of the buildings is between 10–40 m for low rise and 20–30 m for high-rise. The distribution of b and d of the calculated cases is given in Figure 12.a. For reference, we also plotted the dimensions of several CLT and other timber buildings collected from the literature. Two planned concrete tower buildings from Finland are given with green triangles. These two buildings are only studies.

- The mass density of the structures is given in Figure 12.b. It is assumed that low-rise buildings can have low mass density (50–180 kg/m^3), but with a certain height, the mass density will approach the mass density of concrete buildings (120–240 kg/m^3). Again, some real cases are included in the figure as reference.

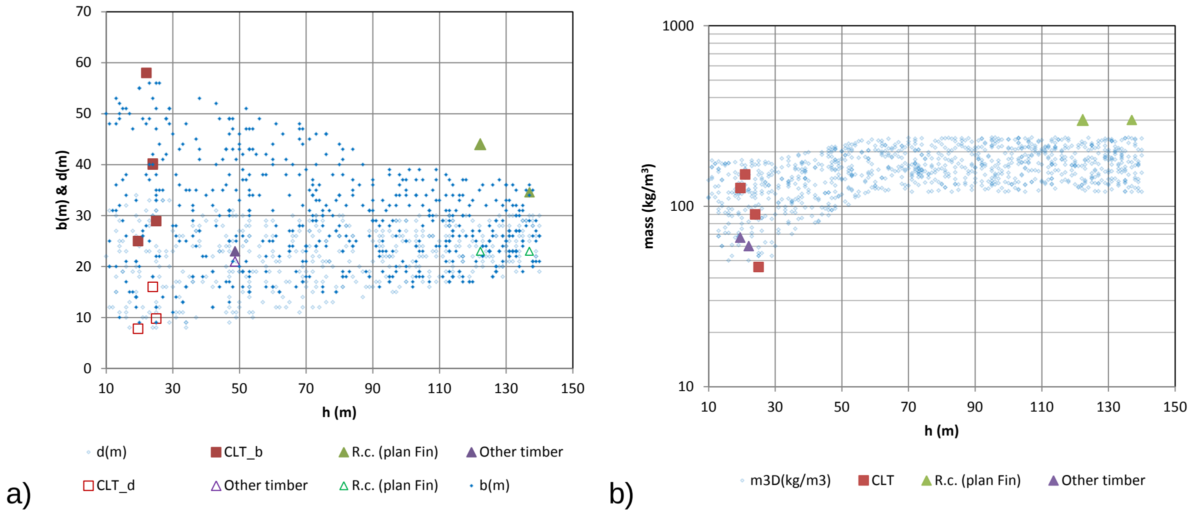


Figure 12. Width/depth (a) and mass density (b) distribution of the calculated buildings

- The natural period of the building roughly conforms to the simplified equation of Eurocode (Equation 17). The different options have been compared in Figure 13. It appears that, measured values for CLT and timber building fit well with the steel structures proposal of Tamura, which is essentially identical to the equation of EN1991-1-4 (Equation 17). The depth dependent ECCS (1987) proposal seems to be less sensitive, especially for CLT buildings (Figure 13.b, green rectangles).

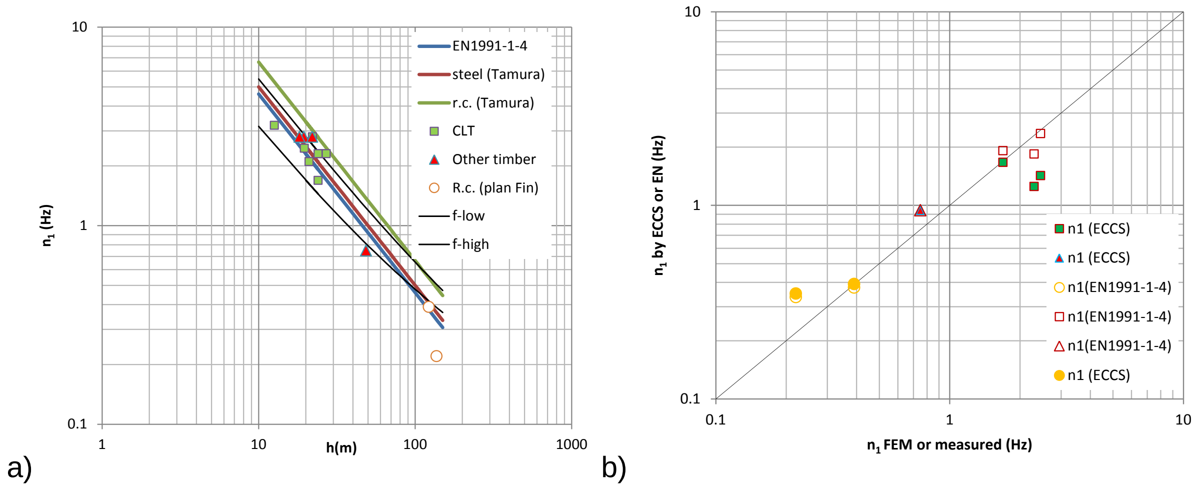


Figure 13. Options for natural frequency estimates, and their fit with FEM and measured values for timber buildings (a) and comparison of the ECCS and EN1991-1-4 predictions of the natural frequency (b),

- Characteristic 10 minutes mean wind velocity is between 21 and 26m/s, in order to cover the zones in Finland.
- Wind load is corrected using the probability coefficient (Equation 2) and the 1 year return period value is used for estimating vibrations. This effectively introduces a reduction of winds by a factor of 0.749.
- The force coefficient (c_f) for the buildings was back calculated from pressure, suction and friction coefficients. For h/d > 5, EN1991-1-4 Table 7.1 does not provide specific ways to calculate pressure coefficients. However, both the British NA (UK/NA, 2010) and the planned Finnish NA draft (Ympäristö, 2016) suggest that for these cases the values corresponding to h/d = 5 should be used. For h/d > 5 the force coefficient (c_f) could also be evaluated using the direct method (see Clause 7.2.2 Note2, and Table 7.16 of EN1991-1-4). However, this would result in a discrepancy in the c_f values for the two groups. The values of c_f used in this study are shown in Figure 14, compared with the force coefficients suggested for rectangular elements (Figure 7.23 of EN1991-1-4).

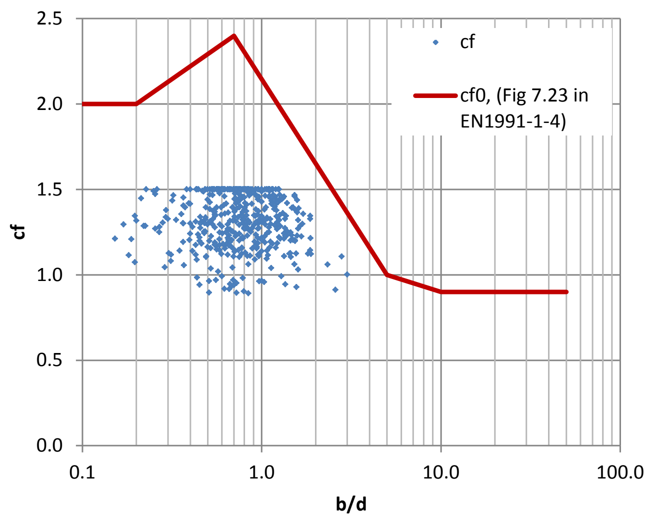


Figure 14. Force coefficients used for the calculation cases

- When the force coefficient (c_f) was calculated from pressures, the distribution of pressures was not detailed with height. Instead, conservatively the pressure at the highest point in the structure was assumed.

- In the calculations carried out here the effect of friction was negligible (A_{fr} = 0). Only about 2–3% of the buildings had enough depth (d) in the wind direction to even require a friction force calculation. The highest contributions of friction forces were 15–20% of the cumulated pressure-suction forces. Friction coefficient was taken 0.02 corresponding to rough lateral surfaces (Table 7.10, EN1991-1-4).

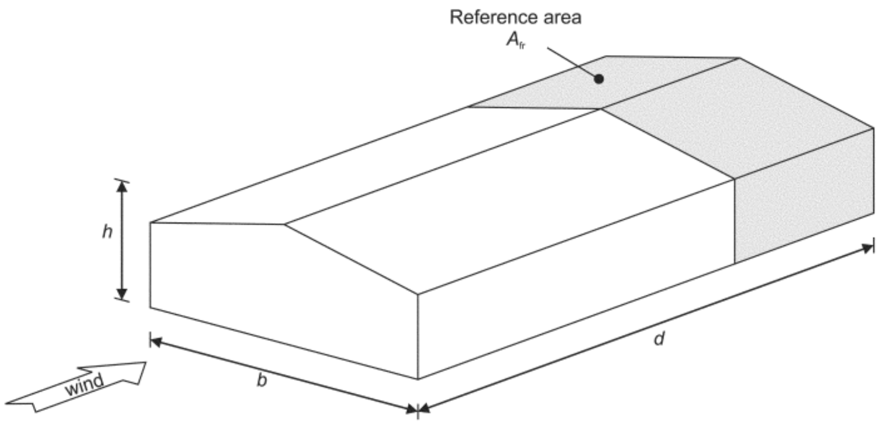


Figure 15. Definition of reference area for friction (A_{fr}).

- Orography factor was taken 1.

- Structural damping was taken in the range of ξ_s = 0.012–0.036.

- Annex C of EN1991-1-4 was used to calculate the wind direction accelerations, presuming uniform horizontal mode shape (G_y = 1/2, K_y = 1) and linear mode shape with the height of the building (G_z = 3/8, K_z = 3/2). This probably fits better the low to medium range cases.

This procedure was benchmarked with a few cases of Steenberger et al. (2009) given in Table 1. Specifically the h = 200, 150 and 100 m buildings were recalculated with z_0 = 0.01, with parabolic and linear mode distribution. As it can be observed in Table 2, results are reasonably agreeing, with a degree of conservativeness.

Table 2. Acceleration estimates for h = 100, 150, 200 m buildings. Comparison of calculation sheet developed with values given by Steenberger.

Height	n (Hz)	Mode shape	a.proc2 (m/s^2)	Our calculation (m/s^2)	Error
200	0.2	linear	0.042	0.047	11.9%
		parabolic	0.053	0.060	13.2%
150	0.3	linear	0.031	0.033	6.5%
		parabolic	0.039	0.042	7.6%
100	0.4	linear	0.026	0.025	3.8%
		parabolic	0.033	0.033	0%

The accelerations calculated with the EN1991-1-4 (2005) methodology have been compared with the simplified formula proposed by Carpenter et al (2013). The velocity *V(h)* is introduced in m/s, frequency in Hz and mass m_e in kg/m, and the acceleration a_{top} is received in m/s^2.

$$a_{top} = 0.46 \cdot \frac{V^3(h)}{m_e \cdot n_1} \qquad 19$$

The agreement of the results is remarkable considering the simplicity of the formulation compared with the full EN 1991-1-4 methodology. For each building, the difference between the accelerations calculated with the two methods can be estimated by the distance of the blue dots from the diagonal line of the plot (Figure 16). It has also to be noted that the wind velocity *V(h)*, used by Carpenter was the one hour mean wind speed at the top of the building, so not fully compatible with the values available in the Eurocode. Therefore the formula was applied using $V_m(h)$, the mean 600s wind speed at the top of the building. The largest relative error observed on the data was 5.9 and the average relative error about 1.3.

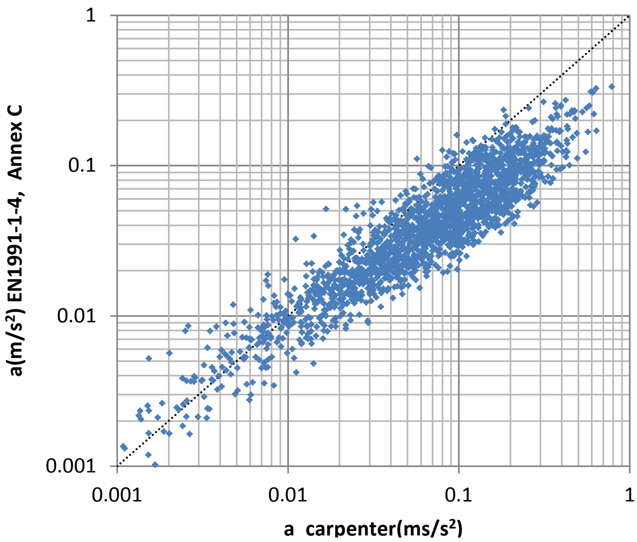


Figure 16. Agreement between the formula given by Carpenter et al (2013) and the EN1991-1-4 procedure. Blue dots are calculation points for a certain building with the two methods.

A better agreement with the EN1991-1-4 method can be achieved using the simplified equation below (Figure 17). In this equation the velocity is m/s, the mass is kg/m, frequency (n_1) in Hz, and the structural damping ration (ξ_s), which is non-dimensional, was introduced. The largest relative error of this formula is 100–160% with an average error of 20–25%.

$$a_{top} = 0.5 \cdot \frac{V_m^{1.5}(h)}{m_e \cdot n_1 \cdot \xi_s} \qquad 20$$

As discussed earlier the conditions of use for the formula are:

- Rectangular buildings. Height 10–140 m; width and depth as given in Figure 12;
- Mass density in the range given in Figure 12;
- Characteristic 10 minutes mean wind velocity between 21 and 26 m/s;
- Wind load using 1 year return period wind;
- Orography factor 1;
- Structural damping in range of ξ_s = 0.012 - 0.036.

- Uniform horizontal and linear vertical mode shape is reasonable assumption;

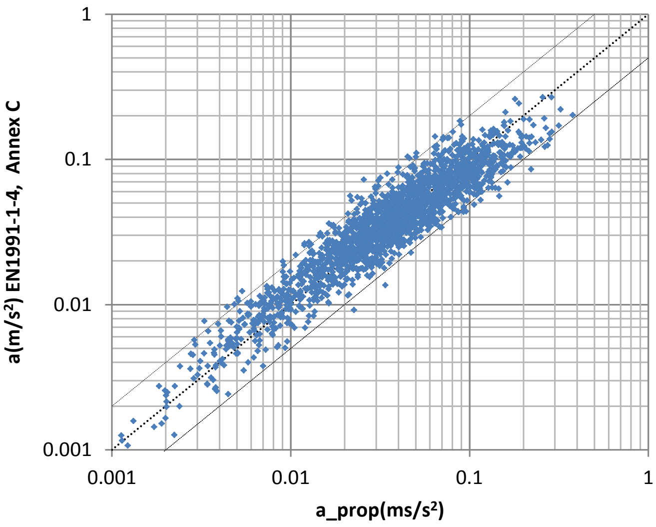


Figure 17. Agreement between the proposed formulation for CLT buildings (Equation 20) and the EN1991-1-4 procedure. The dashed lines show the limitation of two times, respectively 0.5 times deviation (i.e. 100% error).

It has to be noted that the structural damping ratio was introduced in order not to require the calculation of the aerodynamic damping. The equation should not be used for structures with devices specially designed to reduce building vibrations (i.e. dampers), the calibration range being ξ_s = 0.012–0.036, as mentioned earlier. It was also noticed that in this damping range the effect of damping is not significant. Similarly precise prediction can be achieved with a formula which ignores the damping altogether (containing only to Carpenter's parameters). However, it is clear that damping must have an effect on vibrations, so the term was retained in order to respect the physical means of vibration propagation.

It should also be noticed that floor accelerations are inversely proportional with the building mass (Equation 20). Hence, overestimating the mass will result in vibration reduction (i.e. un-conservative design). Especially, it should be considered carefully what proportion of the live load is realistic to be used in the wind vibration calculation, an SLS combination. But in CLT buildings, the variation of humidity may also result in a change of ±10% of the structures mass.

In any case, the formulation by Carpenter (Equation 19) and the proposed formula in Equation 20 provide the same acceleration value for v_m = 21 m/s when the damping in 0.0113 (1.13%) and for v_m = 26m/s when damping ration is 0.0082 (0.82%).

5. Determination of acceptance levels of vibrations

Vibrating buildings can cause unsafe feelings of the occupant. Therefore, with respect to comfort, limit values for the vibration intensity shall be known.

5.1 EN 1990 (2002)

Annex A1 of the standard requires in that verification of serviceability limit should also concern the vibrations that can cause discomfort to people. Possible sources of vibration that should be considered include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These, and other sources, should be specified for each project and agreed with the client. Concerning to the analysis of the dynamic response of the structure the standards EN1991-1-1 (2002), EN1991-1-4 (2005) and ISO 10137 (2007) are referred.

5.2 ISO 10137 (2007)

The standard gives recommendations on the evaluation of serviceability against vibrations of buildings. Its informative Annex D give guidance for human response to wind-induced motions in buildings. Horizontal peak accelerations of building with a one-year return period are applied to the evaluation of habitability. The Annex gives evaluation curves for acceptable horizontal motions. The peak accelerations of the target floor should not exceed the basic evaluation curve for the respective occupancy (Figure 18). The level of the evaluation curve for 'residences' comes to 2/3 of the curve for 'offices'. The resultant curve for residence is close to the 90% level of the perception probability.

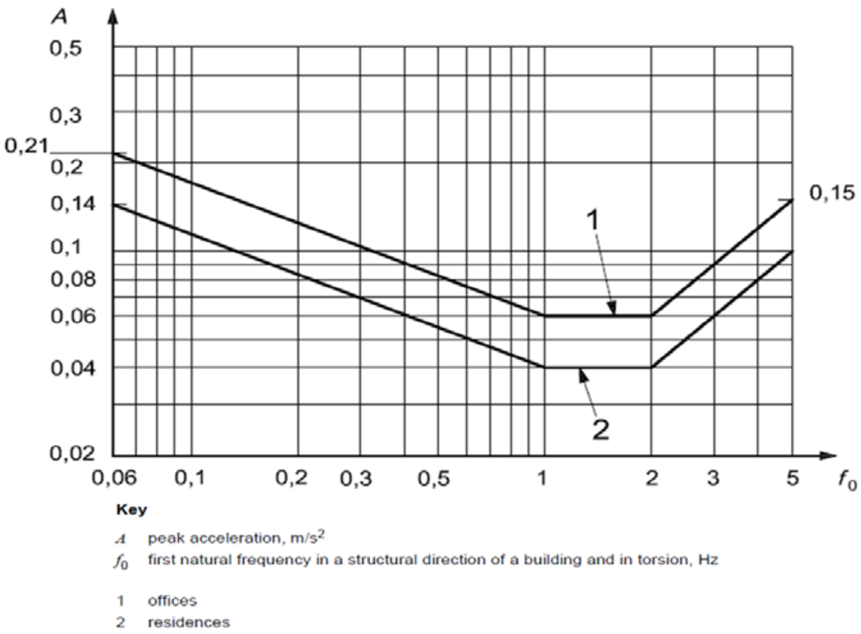


Figure 18. Limits for horizontal peak accelerations in ISO 10137 (2007).

5.3 EN-ISO 8041 (2005)

The standard gives guidance for measuring and instrumentation. It does not give guidelines for the vibration limits. The vibration magnitude is expressed by using frequency-weighted root-mean square (rms) acceleration. Frequency weightings are given for different applications. Frequency weighting curve W_d (Figure 19) is given for horizontal whole-body vibration.

For a harmonic oscillation the weighting is inverse to the limit vibration curve normalized to its maximum value, therefore ISO 10137 weighting can be compared with W_d weighting. Figure 19 shows that for frequencies greater ~0.3 Hz the curves are practically equal. Then the limit amplitude 0.04 m/s² in ISO 4354 corresponds W_d weighted acceleration of 0,028 m/s² (= 0.04 m/s² /√2).

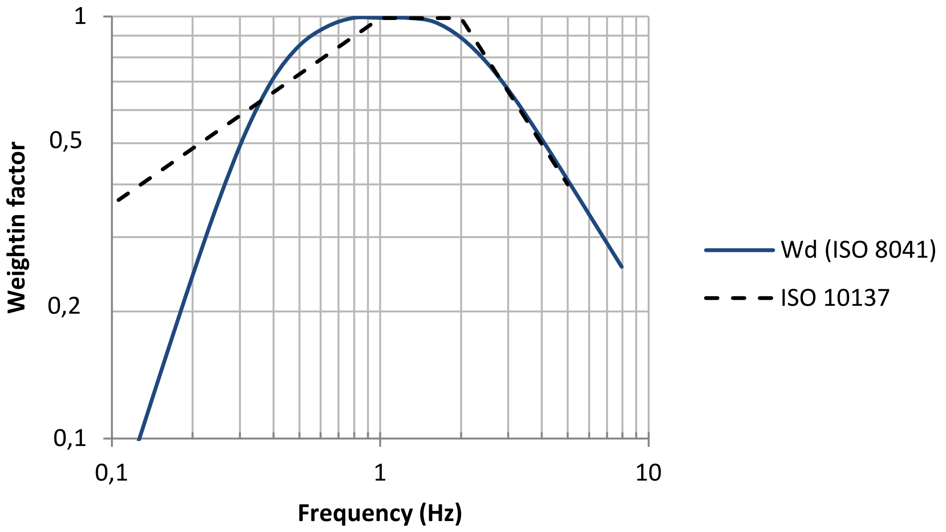


Figure 19. Weighting curve W_d in EN-ISO 8041 (2005) compared with ISO 10137 weighting.

5.4 ISO 6897 (1984)

ISO 10137 (2007) refers also to ISO 6897 (1984) when the vibration of the building is in frequency range 0.063 Hz to 1 Hz. The Annex of ISO 6897 gives limit values of rms accelerations for buildings used for general purposes. The limits are based on the worst 10 consecutive minutes of wind storm with a return period of at least 5 years. Suggested limit curve shown by peak accelerations (rms values multiplied by √2) are shown in Figure 20. The limit curve is practically identical with ISO 10137. Figure 20 shows also the lower and average threshold of perception of horizontal motion of normal adult population (ISO 6897).

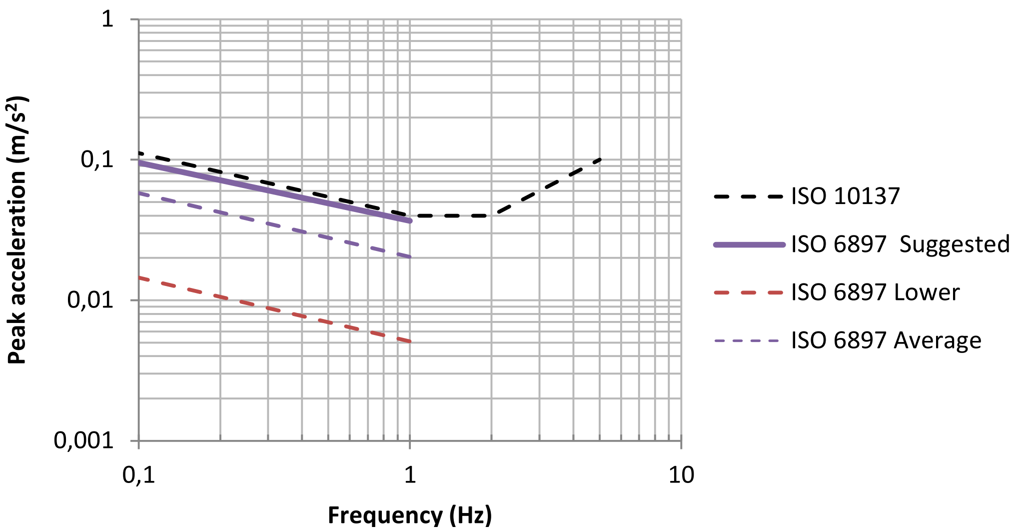


Figure 20. Suggested limits for horizontal peak accelerations in ISO 6897 compared with ISO 10137 (residences).

Annex of ISO 6897 remarks, that if a building is subject to even extremely small oscillations, visual effects would exaggerate the sensation of motion and the satisfactory magnitudes of acceleration would be less than suggested for general purposes.

5.5 AIJ (2004) guidelines in Japan

Instead of giving one recommended line, AIJ guidelines gives five curves: H-10, H-30, H-50, H-70 and H-90, as shown in Figure 21. The number of each curve indicates the perception probability as a percentage, i.e. 10% of people can perceive the vibration specified by the H-10 curve. It can be noticed from Figure 21 that ISO 10137 curve for residential building corresponds quite well to H90 curve in AIJ guidelines. The criteria for building habitability to vibration should be decided by a building owner. The basic evaluation curve is specified with the first natural frequency of the building. Horizontal vibrations of buildings with one-year return period are applied to the evaluation of habitability (Tamura et al. 2004).

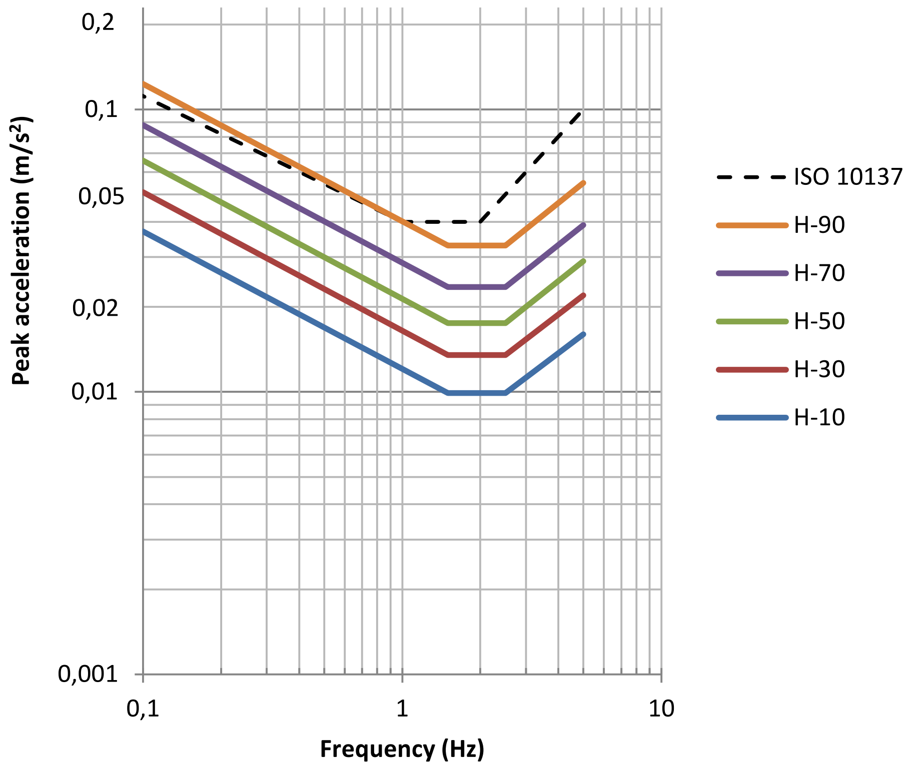


Figure 21. AIJ probabilistic limit curves compared with ISO 10137 curve (residences).

AIJ curves are connecting earlier research results for the low-frequency region from 0.125 Hz to 0.33 Hz and the high frequency region from 1.0 Hz to 6.0 Hz (Figure 22, left image). These experiments were made basically for uniaxial sinusoidal motions, but tests were also conducted for random motions and for bilateral elliptic motions simulating wind-induced building motions. Randomness did not seem to affect the perception threshold. The perception threshold for random motions was almost the same as that for sinusoidal motions. Thus, the motion perception for wind-induced vibration of a building might be simply based on the acceleration amplitude and its predominant natural frequency (Tamura 2009).

As Annex of ISO 6807 mentions, visual effects would exaggerate the sensation of motion and the tolerated magnitudes of acceleration would be less than suggested for general purposes. Probabilistic perception thresholds in Figure 22 (right image) show the motion perception probability only from visual cues inside rooms considering the probability of seeing visual cues. In the low frequency region less than 2 Hz, probabilistic visual perception threshold can be lower than the probabilistic perception threshold by only body sensation (Tamura 2009).

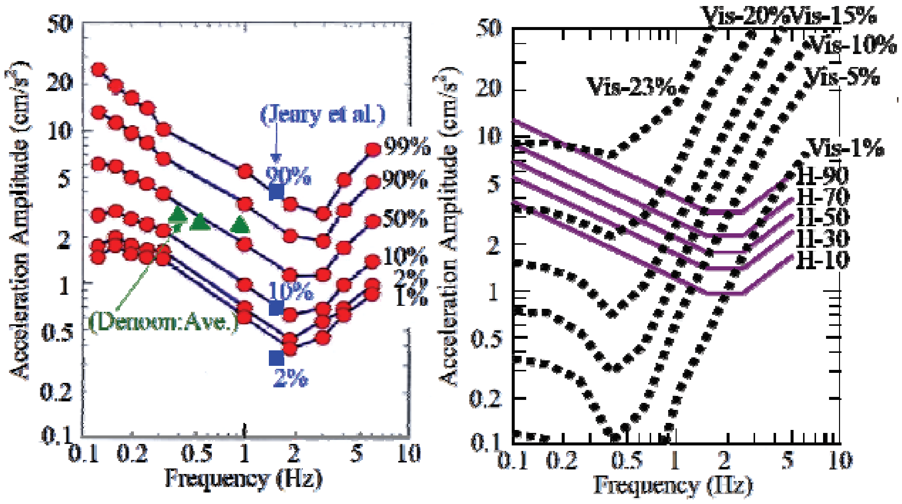


Figure 22. Left: Perception threshold by only body sensation. Right: Exceptional threshold by only visual cues and comparison to AIJ guidelines (Tamura 2009).

5.6 Traffic and human-induced vibrations in Finland

VTT's guideline for comfort-based traffic-induced vibration (Talja 2011) is using the W_m weighted (SFS-EN ISO 8041) rms vibration. The proposed vibration limit is 0,011 m/s^2 for new residential areas and 0.022 m/s^2 for old residential areas. These values correspond to weighted rms velocities of 0.3 mm/s and 0.6 mm/s. For offices double values are allowed. For a harmonic oscillation, the limits can be converted to vibration curves as shown in Figure 23. The vibration level of ISO 10137 matches quite well the curve 'old residential' at frequency range 0.5-3 Hz. It should be noted that in the case of traffic-induced vibrations only the frequencies greater than 1 Hz are under consideration, and the most important frequency range is 5-20 Hz.

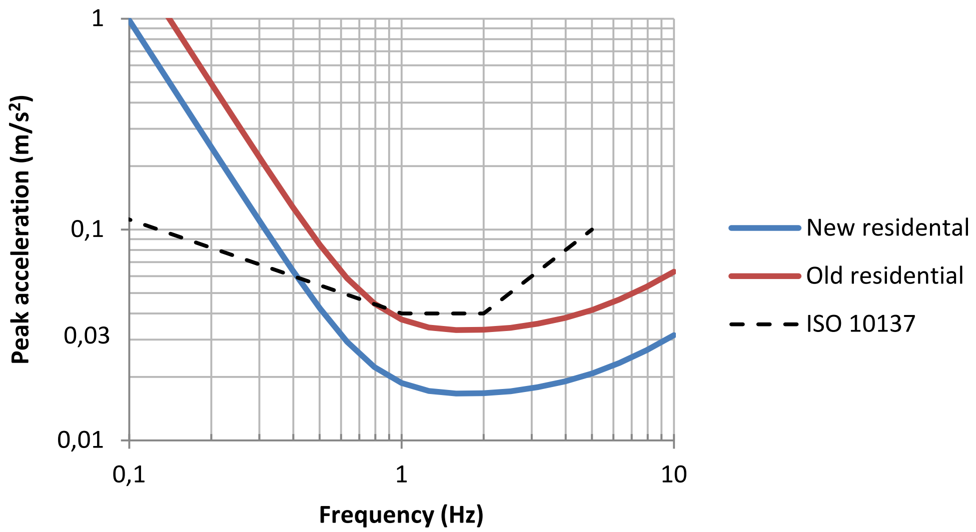


Figure 23. Proposed limit vibration for traffic-induced vibrations compared with ISO 10137 curve (residences).

According to Finnish code of practice (TRY 2005, normal class) the limit peak acceleration is 0.03 m/s^2 for walking-induced vibrations when the vibrations are induced by neighbours. The value is used for floors with fundamental frequencies of 3–10 Hz. The limit 0.03 m/s^2 is between the curves for 'new residential' and "old residential' in Figure 23. For comparison it can be noted that the limit for self-induced vibrations is 0.075 m/s^2.

5.7 A field study of the effects of wind-induced building motion

One of few field studies of occupant wellbeing and work performance has been carried out by Lamb et al. (2014). In the study 47 office workers on high floors of wind-sensitive buildings and 53 control participants completed 1909 surveys across 8 months in Wellington NZ. The predicted motion perception is classified to categories 'no motion', 'possible' and 'definite'. Figure 24 shows the maximum predicted acceleration for each of 11 building (the black diamonds), where participants reported 'definite' building motion. In the study period, the maximum wind speed reached only 76% of one-year return period (T_R = 1) (Lamb et al. 2014).

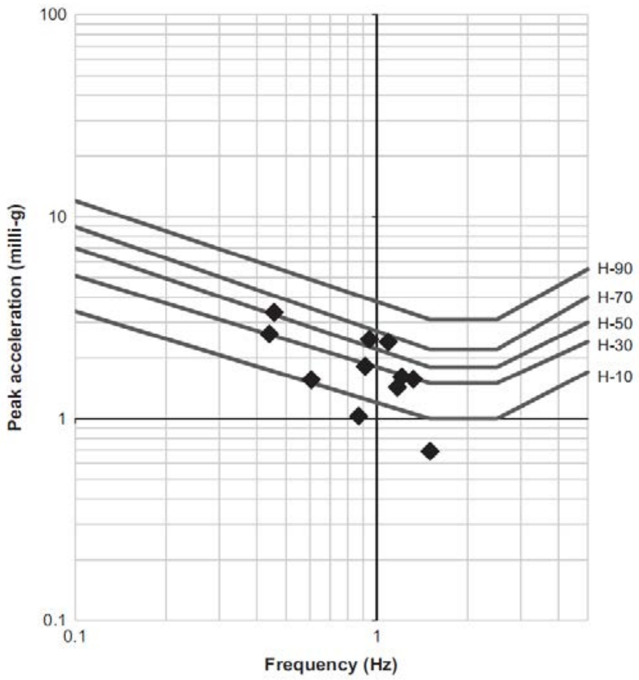


Figure 24. Predicted peak accelerations in 11 study building compared to AIJ guidelines (Lamb et al. 2014).

Lamb et al. (2014) points out that ISO 10137 and AIJ curve H-90 are both based on the level at which 90% of occupants are likely to perceive motion, leaving a comfort factor to the discretion of the building designer. However, below the 90% threshold, a significant proportion of occupants will already perceive motion and are likely suffering from the associated effects. Most of the effects are observed at around the AIJ H-30 to H-50 curves. The results show that the effects of building motion are emergent, which mostly manifest as symptoms of sopite syndrome (tiredness, low motivation, distraction from work activities, and low mood), or low-dose motion sickness. Affected individuals attempt to manage their own discomfort, and indicate a preference to work at different location during motion, take 30–40% longer breaks, and attempt to self-medicate using analgesics. Humans are adaptable, allowing most occupants to continue their work activities, but at reduced levels of performance and comfort. Design criteria for tall buildings should attempt to minimise the effect of building motion on work performance and wellbeing, rather than focus only on motion tolerance or formal complaint to building owners (Lamb et al. 2014).

6. Conclusions and summary

Based on the study of the available European design standard for evaluation of vibrations in structures, we can conclude that wind direction vibration can be estimated using calculations. These calculations can be substantially simplified, and even very simple formulations can be developed for restricted building types. We suggest the use of Equation 20 for an explorative evaluation of the wind direction accelerations, with the notice that the application range is observed (conditions for parameters).

A better calibration for such simplified equation for CLT buildings could be made if the dynamic properties of the typical buildings are determined. At present we used international literature data for calibrating natural frequency, damping, masses etc. Better parameters can be obtained by reviewing designs of existing buildings in Finland and possibly, by dynamic measurements on finished structures. Natural frequency and damping could be estimated better, but also direct correlations of the measured wind-speed and acceleration can be done. Hence, calibrating Equation 20 with measurements could be carried out in similar fashion as in Carpenter et al, (2013) and Flay et al. (2013).

Another aspect for vibration estimation concerns other phenomena than wind-direction vibration interacting with the first natural mode. Some torsion is always present in real structures even if they are nominally symmetric. Cross-wind vibration may be important, and not covered by the current calculations. In fact EN1991-1-4 is not providing methodologies for the topics mentioned.

When we come to the vibration limits, it has to be remembered that human perception of motion and tolerance of wind-induced tall building vibration are essentially a subjective assessment. Hence, there is currently no single internationally accepted occupant comfort criteria for satisfactory levels of wind-induced vibration in tall buildings. Human response to motion, particularly wind-induced building motion, is a complex mix of a variety of psychological and physiological factors. Furthermore, in the assessment of occupant comfort in wind-excited buildings, the acceptance criteria should be based on occupant comfort and general well-being, and not only on occupant perception of the motion. Also the societal influences (culture, ethnicity) may play a role in defining motion acceptability (Kwok 2013).

Based on the concise literature study and comparisons with other reference values used in Finland, ISO 10137 (2007) provides a good basis for the assessment of discomfort-based wind vibrations (Figure 18). There the evaluation curves for acceptable horizontal motions are given for offices and residences. In the frequency range suitable for comparison (greater than 1 Hz), ISO curve for residences matches quite well the curve ‘old residential’ recommended for traffic-induced vibrations (Talja 2011) (Figure 23). Therefore it may be best suited for the evaluation of the vibration of the old buildings. For design of new buildings it is justified to use only half the values of the curve. Then the limit corresponds quite well to H-50/H70 probability curve in AIJ (2004) guidelines (Figure 21). The latest field studies (Lamb et al. 2014) on occupant wellbeing and work performance support this proposal. Also ISO 2631-2 (2003) states that experience in many countries has shown that adverse comments in residential situations may arise from occupants of buildings when the vibration magnitudes are only slightly in excess of perception levels.

In the measurement of wind-induced vibrations EN-ISO 8041 (2005) standard is recommended to be used when the dominating frequencies are greater than 0.3 Hz. Then vibration magnitude is expressed by using frequency-weighted rms acceleration. Frequency weighting curve W_d (Figure 19) for horizontal whole-body vibration should be used. Then for a harmonic oscillation the rms value of 0.028 m/s^2 corresponds to ISO limit curve for residences (Figure 19). However, in the design of new buildings the target limit value should be half of that. The time averaged weighted rms acceleration is determined by 8 seconds time window (integration time for running averaging).

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