ACTUAL PROBLEMS OF THE WIND ACTIONS ON STRUCTURES AND APPLICATION OF EUROCODES

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Abstract: In accordance with implementing EN 1991-1-4 to Slovak National Standards it is necessary to take into account aero-elastic instabilities and resonant response due to wind actions. The pressures or forces whose effects are equivalent to the extreme effects of the turbulent wind represent the wind action. The effect of the wind on the structures as well as the response of the slender structures depend on the size, shape and dynamic properties of the structures, orography and meteorological data, type of terrain and reference height. Dynamic response should be considered for flexible structures like masts, chimneys, towers, open frames or canopy roofs, and structural elements of open lattice structures, cables.

Keywords: Wind velocity, resonance, aerodynamic coefficient, wind amplitudes, wind pressure, aero-elastic instabilities, turbulence

1 Introduction

The recent years were accompanied by the rise of the wind storms and height of wind velocity. Therefore it becomes necessary to consider also dynamic wind effect and possible resonance response for the structures, which are sensitive to the wind action and the wind is dominant design factor, as for example light halls, canopy roofs, free standing walls, signboards, cables, light masts, chimneys, towers, open frames and structural elements of open lattice structures, footbridges.
The basic parameter for determination of the wind load of the structure is peak velocity pressure q_p, which contains mean wind velocity and fluctuating part of wind velocity- turbulence. This is influenced by the atmospheric conditions in the given locality, height above terrain and local influences, as for example roughness and orography, season and wind direction. The peak velocity pressure is calculated as the pressure in conditions of the mean wind speed and short-term velocity fluctuations. The intensity of turbulence near the ground according to the Euro codes reaches up to 42%. The overall response of the structure is regarded as the superposition of two components – quasi-static and resonant. For the most of structures the resonant components are negligible and they can be counted by the dynamic coefficient.
The special attention must be paid to the phenomena of aero elastic instability as stochastic and resonance response, vortex shedding, divergence, galloping and flutter, which may be observed on the slender light and flexible structures.
EN 1991-1- 4 with national annex STN EN 1991-1-4/NA give guidance on the determination of natural wind action, which take into account specific Nationally Determined Parameters to be used for the design in Slovakia. Wind action on the structure is dependent on the location and quality of meteorological data, terrain category, etc.

2 Wind action on light structures situated near the ground

Due to big influence of turbulence near the ground, light steel and wooden structures are exposed to bigger gust of wind.
Recommended rule for determination of peak velocity pressure (1) according to EN 1991-1-4 is:

$$q_p(z) = \left[1 + 7I_v(z)\right] \cdot 1/2 \cdot \rho \cdot v_m^2(z) = c_e(z) \cdot q_b \qquad (1)$$

The values of peak velocity pressure are much higher than according to STN 73 0035.
The intensity of turbulence is defined:

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_o(z) \cdot ln(z/z_0)} \qquad (2)$$

The turbulent intensity at height z is the standard deviation of turbulence divided by mean wind velocity. Value near the ground for different terrain is between 22 – 42 % (see Fig.1)

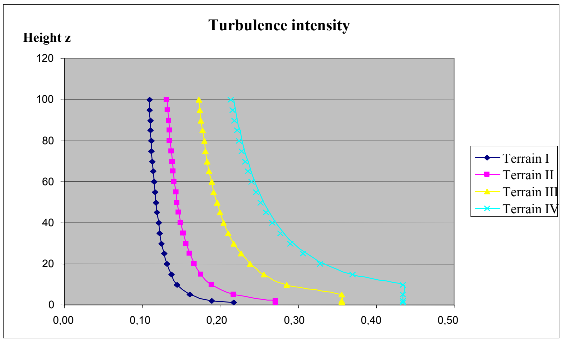


Fig. 1 The turbulence intensity as a function of height and terrain category

Comparison of internal forces due to wind action on the light steel hall (see Fig.2), which is situated in terrain category III (subtopia) was calculated according to EN 1991-1-4 and also according to STN 73 0035 is illustrated in diagram in the Fig.3.

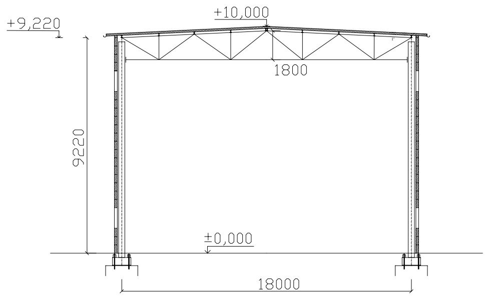
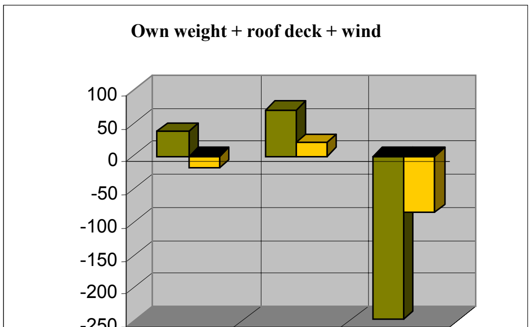


Fig.2 Light steel hall

From the diagram in the Fig. 3 we can see, that the Euro code is much stronger for wind action on the structures than STN.



	N_{max}	V_{max}	M_{max}
EN	36,275	69,708	-247,071
STN	-17,764	21,301	-85,338

Fig.3 Comparison of wind load due to EN 1991-1-4 and STN 73 0035

It is caused especially by higher value of wind pressure, which take into account intensity of turbulence also for quasi-static calculation (peak velocity pressure at height 10m, calculated by EN 1991-1- 4 gives value $q_p(z = 10m) = 0{,}993 kN.m^{-2}$, this value compares to the same part of wind load $w_o.\kappa_w = 0{,}363$ kN.m^{-2} due to STN 73 0035 is 274% greater). Also the wind action on the roof of the hall is different for the different codes. Euro-code EN 1991-1-4 divides roof into five zones with different pressure coefficients, compared to code STN 73 0035, which divides area of the roof only in two zones. The biggest differences are manifested in bending moments (Fig.3).

2.1 Free-standing walls

Results of experimental measurements in the wind tunnel give us for free standing walls, parapets, fences different resulting pressure coefficients. Coefficients are specified for zones A, B,C and D (see Fig.4). The highest values of the wind pressure were observed near the corner, for the wind direction 45 °.

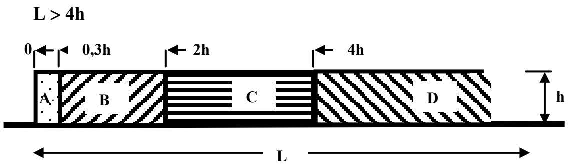


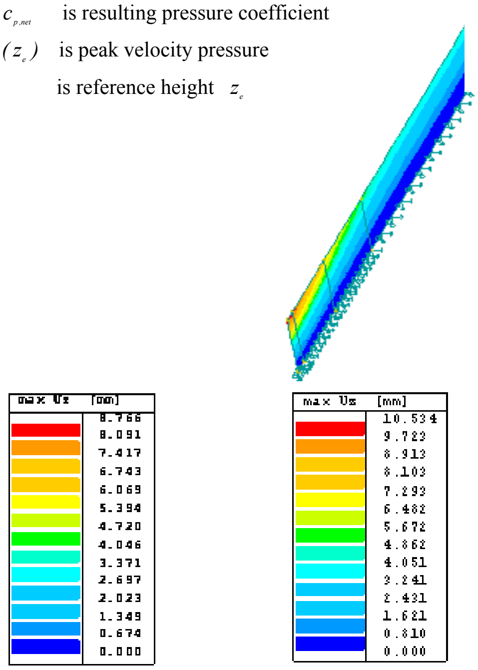
Fig. 4 Zones of free standing walls with different resulting pressure coefficients

$$w_{net} = c_{p,net} \cdot q_p(z_e) \tag{3}$$

where $c_{p,net}$ is resulting pressure coefficient

$q_p(z_e)$ is peak velocity pressure

is reference height z_e



a/ Terrain category IV (city centers)

b/ Terrain category II (area with low vegetation and isolated buildings)

Fig. 5 Influence of wind effect in the zone A for free standing concrete wall

The values of resulting wind coefficients $c_{p,net}$ depend on the solidity ratio φ, and geometry of structures (L/h) and from the corners of the walls. The highest value of the wind pressure according to EN 1994-1-4 is in the zone A (see Fig. 4). Influence of the zone A for different terrain category can be seen in the Fig.5, where are presented the values of the horizontal deflection due to turbulent wind actions. Numerical application was calculated for full concrete wall: φ=1, h = 5 m and L = 50m (Fig. 5). Fundamental wind velocity was v_b = 26 m/s. For the free-standing walls and parapets with solidity ratio φ = 0,8 are the pressure coefficients smaller and the same /equal/identical for all zones A,B,C,D.

3 Wind actions on the free standing canopy roofs

The response of structures due to wind actions can be observed on the canopy roofs – structures that do not have permanent walls, such as stadiums, petrol stations, dutch barns, etc. The wind distribution over frequencies is according to experimental measurements for the sport stadium (see 6. and Fig.6) expressed by the non-dimensional power spectral density function (Fig.7).

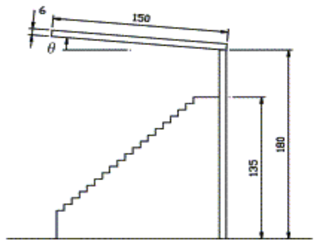


Fig. 6 Light open canopy roof (sports stadium)

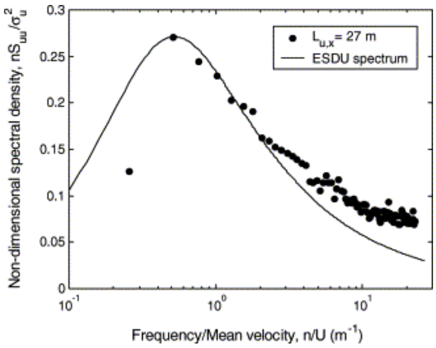
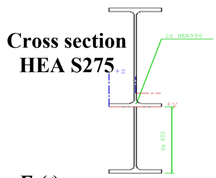


Fig. 7 Power spectra density function

The wind load is presented either as a wind pressure or a wind force. For the steel hall in Fig. 8, we calculated dynamic response due to eccentric wind force $F_w(t)$ (see part 7.3 in 1.):

$$F_w(t) = c_{F,net} \cdot \frac{1}{2} \rho \cdot v^2 \sin(\omega_i \cdot t) \cdot A_{ref} \tag{4}$$

Harmonic wind force by vibration in resonance with 2nd natural mode (n_2 = 5,101 Hz) was considered normal to the roof with eccentricity d/4 of axis cross-section.
Second natural frequency is situated near to power spectra function peak.



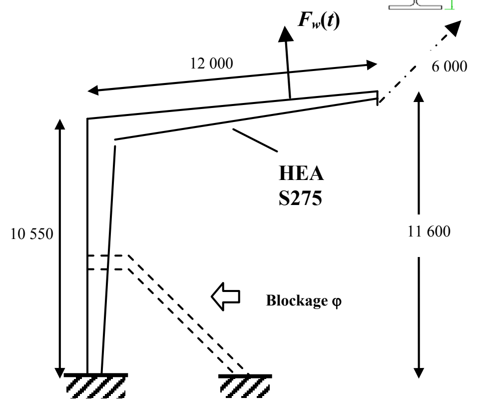


Fig. 8 Open sports stadium roof in Bratislava

Maximal amplitudes of the free end of the roof for the mean wind velocity 10,2 m/s depends on the degree of blockage under canopy. For the empty canopy the max displacement is 0,044 m and for the fully blocked roof the value is 0,068 m (see 8.). Generally, we can say that on the preliminary stage of design of canopy roofs it is possible to find the first and second natural mode out of the maximum power spectra density function and response and minimize wind response.

4 Vortex shedding

The aero-elastic instabilities were observed on large stacks, antennas and light masts, towers, bridge hangers and supports. Vortex shedding should be investigated when the ratio of the largest to the smallest crosswind dimension of the structure (in the plane perpendicular to the wind) exceeds 6. If the cylinder is flexible in cross flow, and natural frequency is near to vortex shedding frequency, exciting force comes into resonance with vibration of cylinder and this effect is vortex resonance. The vortex shedding is highly Reynolds number dependent. There are three typical ranges of flow separation.

1/ subcritical $Re < 2{,}3\ .10^5$
2/ critical and postcritical $2{,}3\ .10^5 < Re < 3{,}5{,}5\ .10^6$
3/ transcritical $5\ .10^6 < Re$

In subcritical and transcritical ranges the flow separation becomes more regular. The exciting force increases and starts vortex-excited vibrations with vortex shedding frequency n:

$$n = \frac{St \cdot v}{d} \qquad (5)$$

From equation (5) we can calculate „critical wind speed", when the vortex shedding may occur:

$$v_{crit} = \frac{n_{(i),y} \cdot b}{St} \qquad (6)$$

where b - width of the structure (perpendicular to the wind direction) for circular cylinder outer diameter
$n_{(i),y}$ - is fundamental frequency of cross-wind vibration of the mode i in [Hz],
St – Strouhal number

4.1 Calculation of the peak amplitude according to EN 1991-1-4

The largest displacement according to E.1.5.2 in 1.is:

$$\frac{y_{F,max}}{b} = K_w \cdot K \cdot c_{lat} \cdot \frac{1}{St^2} \cdot \frac{1}{Sc} \qquad (7)$$

where K_w is correlation length factor (E8) in 1.:

$$K_w = \frac{\int^{L_j} \Phi_{i,y}(z)dz}{\int_0^{l} \Phi_{i,y}(z)dz} \leq 0{,}6 \qquad (8)$$

Constant of the mode shape K (E9) in 1.:

$$K = \frac{\int_0^{l} \Phi_{i,y}(z)dz}{4\pi \int_0^{l} \Phi^2_{i,y}(z)dz} \qquad (9)$$

The exciting lift coefficient c_{lat} depends on Reynolds number (see Tab.E.2 and Fig.E.2 in 1.). Scruton number (see E.4 in 1.):

$$Sc = \frac{2 \cdot m_{i,e} \cdot \delta_s}{\rho \cdot b^2} \qquad (10)$$

Where ρ =1,25 kg/m^3 is air density,
$m_{i,e}$ is equivalent mass per unit length,
$\Phi_{i,y}$ is the mode shape i.

In the Tab.1 there are the structures and their main characteristics sensitive to vortex shedding (see 4.).

Tab.1

Structure.	m_e [kg/m´]	h/b	$v_{crit,1,2}$ [m/s]	Sc	Re	δ_s
Steel stack I	225	25/1,3	14	2,56	$1{,}2.10^6$	0,012
Steel stack II	442	50/1,8	6,48	2,5	$0{,}7.10^6$	0,012
Concrete chimney	9800	120/4,6	7,2	21,8	$2{,}4.10^6$	0,03
Steel light masts	31	20/0,49÷ 0,25	1,23 7,64	11,0	$2{,}05.10^4$ $1{,}78.10^5$	0,012

In the Tab.2 the maximal resonance displacements in cross wind direction due to vortex shedding are compared (see 4.). Vortex resonance responses were calculated according to (7) for slender and wind sensitive structures. In this case the EN 1991-1-4 is less strict, because vortex shedding mechanism is not uniformly distributed along cylinder axis. The exciting force acts on the correlation length, for the cantilever this length is near the top and increases with increasing amplitude. Comparison of maximal vortex amplitudes due to different codes is in Tab.2

Tab.2

Structure	EN 1991-1-4	Numerical analyses STN 73 0035
	$y_{F,max(1)}$ [m]	y_{max} [m]
Steel stack I	0,249	0,258
Steel stack II	0,265	0,287
Concrete chimney	0,077	0,0795
Steel light masts	0,02	0,0288

5 Conclusion and evaluation

The paper shows that detailed analysis of the wind action, especially for the slender structures and structures with atypical shapes situated near the ground helps us to better understand the wind flow. Results and conclusion in these cases show that it is necessary to take into account stochastic and resonance effect of turbulence intensity. The full dynamical evaluation of a response of sensitive structures is significant for engineer. For structures, which are sensitive to dynamic effects, the calculation procedure could consider critical winds velocities between 5 ÷ 20 m/s (see 2.). To obtain the wind load and response information for the large and slender structures and for the structures situated in regions with special terrain parameters or very cold and stratified flow conditions the wind tunnel tests with appropriate models of the structure and natural wind provide us the design parameters.

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