Wind Loading for the Design of the Solar Tower

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ABSTRACT: Some 25 years ago Professor Schlaich developed the idea of a power plant which uses the power of the upwind flow in a special chimney. Solar insulation drives the flow. He tested the concept successfully in a comparatively small scale facility in Spain. The obvious advantages of Schlaich's technology have stimulated various project ideas particularly in Australia but as well in Africa. The chimney dimensions range from 1000 m in height and 130 m in diameter up to 1500 m in height and a diameter of 280 m depending on the required power output of 100 to 400 MW. The chimney is basically a cylindrical tube designed as a concrete shell structure. An important component in the structural design is the wind loading. It is the topic of the present paper. The tower height is beyond present experience. Simplified models for the wind load which, based on successful tradition provide safe and economic structures are not applicable. The wind flow in the atmospheric boundary layer, in particular its turbulence must be reconsidered since experimental evidence does not go further than 350m mainly. The authors attempt to develop a set of wind loading data appropriate to the challenge of designing these outstanding structures.

1 INTRODUCTION

A solar tower of 100 MW power output requires a chimney of 1000 m height and a diameter of 130 m [1]. After an existing concept it shall be constructed from reinforced concrete as a cylindrical shell with a circular cross-section.

The impacts due to wind forces and the resulting stresses are dominating loads. A building structure of such a height has not been executed before. Simplified load assumptions based on prior experiences which lead to a safe design are not available.

The flow patterns in strong winds are well investigated in theory and experiment for the lower 300 m of the atmospheric boundary layer. In the Prandtl layer the flow character is controlled from the interaction between layers of different velocities which generates turbulent impulses. There are existing sufficiently accurate and reliable models which are applied to evaluate the static and dynamic flow forces for the stability survey of a structure. In the Prandtl layer the mean wind velocity increases with height above ground according to a boundary layer profile. In the same time the intensity of wind turbulence is decreasing. The Ekman layer is directly super-imposed. Here, the Coriolis effect becomes more important while the turbulent mixing effect of the Prandtl layer is reduced. The mean wind direction deviates from the direction in the mixing layer. There is a lack of meteorological data for height of larger than 300 m above ground. The property of the wind in such a height is to be derived from theoretical considerations. This is especially true for the turbulence structure which controls the dynamic wind forces.

The flow over the surface of the tower is featured by high Reynolds numbers of the order of

$$Re \approx 4 \cdot 10^8 \qquad (1.1)$$

for the assumption of a mean wind speed of 45 m/s and a diameter of 130 m. Such values are approximately achieved in full scale measurements at hyperbolic cooling towers which justify the use of experimentally verified static and dynamic pressure distributions at cooling tower shells for the derivation of the pressure distribution at the solar tower. In this connexion the aerodynamic slenderness of the solar tower will be an important factor. Eventually it has to be included that the basic properties of the fluid, especially its density and viscosity, will change considerably.

2 PROPERTIES OF THE WIND

2.1 *Profile of the mean wind velocity and dynamic pressure*

Harris and Deaves [6] derived a logarithmic formula for the wind profile over height which describes both the Prandtl layer and the Ekman layer. Basic variables are the rougness length z_0 and the thickness δ of the atmospheric boundary layer. The gradient of the velocity profile becomes zero for $z = \delta$.

It is commonly accepted to neglect the variability of air density ρ with building height. In the case of the solar tower ρ decreases by 10% between the ground level and the tower top which will cause a reduction of the wind forces by also 10%. Therefore, such influence is considered.

The thermal stratification in strong winds is approximately neutral and the atmosphere is adiabatic. The air density is $\rho_0 = 1{,}25\ \mathrm{kg/m^3}$ at a pressure of 1000 hPa and a temperature of 0°C. Its dependency on height above ground is

$$\frac{\rho(z)}{\rho_0} = \frac{22000 - z}{22000 + z} \quad (z \text{ in m}) \tag{2.1}$$

which leads to the expression

$$\frac{q_m(z)}{q_m(10)} = \left(\frac{V_m(z)}{V_m(10)}\right)^2 \frac{\rho(z)}{\rho_0} \tag{2.2}$$

for the profile of the dynamic pressure. More practical for calculations is the approximation

$$\frac{V_m(z)}{V_m(10)} = \left(\frac{z}{10}\right)^{\alpha_v} \tag{2.3}$$

with an adjustable exponent α_v of the profile. The exponent has values between $0{,}158 < \alpha_v < 0{,}168$ for heights between $200\ \mathrm{m} < z < 1000\ \mathrm{m}$. In an analogous manner the profile of the dynamic pressure over height

$$\frac{q_m(z)}{q_m(10)} = \left(\frac{z}{10}\right)^{\alpha_q} \tag{2.4}$$

is controlled by a variable exponent between $0{,}310 < \alpha_q < 0{,}316$ for heights between $200\ \mathrm{m} < z < 1000\ \mathrm{m}$ if the dependency on height above ground of the air density is considered.

2.2 *Wind turbulence*

The components of wind turbulence are described through the r.m.s. values of the fluctuations of the velocity in the direction of the mean wind, σ_u, in lateral, σ_v, and in vertical directions, σ_w. Commonly, the r.m.s. values are assumed to be constant in the Prandtl layer. Above of approximately $z = 200$ m the standard deviations are decreasing linearly with increasing height and are zero at the upper bound of the boundary layer at $z = \delta$. Such properties are included for the description of the dynamical forces by means of the turbulence intensity. A model by Deaves and Harris [6] uses the fluid-mechanical quantities thickness of the boundary layer δ, roughness length z_0 and shear velocity u^* as the independent variables, and calibrates the model with Kármán's constant $\kappa = 0{,}4$.

As an alternative the turbulence intensity can be approximated through the ratio of the r.m.s. value of the fluctuations of the velocity and the profile of the mean velocity over height.

$$I_u(z) = I_u(10)\frac{V_{m10}}{V_m(z)}\frac{\sigma_u(z)}{\sigma_{u10}} \tag{2.5}$$

which results into [5]

$$\frac{\sigma_u(z)}{\sigma_{u10}} = \frac{\left(0{,}539 + 0{,}09\ln(z/z_0)\right)^{(1-(z/\delta))^{16}}}{\left(0{,}539 + 0{,}09\ln(10/z_0)\right)^{(1-(10/\delta))^{16}}}\,\frac{1-(z/\delta)}{1-(10/\delta)} \tag{2.6}$$

The linear term $(1 - z/\delta)$ dominates for $z/\delta > 0{,}2$, so that the approximation

$$\frac{\sigma_u(z)}{\sigma_{u10}} = \left(1 - 1{,}1(z/\delta)\right) \quad z<1500\mathrm{m} \tag{2.7}$$

is used. Analogous equations can be built up for the components v and w.

3 TURBULENT WIND LOADS AT THE CIRCULAR, CYLINDRICAL SHELL

3.1 *Static, quasi-static und resonant load components*

A structural response generated by the stochastic wind pressures consists of a static part which is constant in time, a quasi-static part containing low frequencies, and resonant fluctuations in the range of the eigenfrequencies of the structure. Time averaged pressure distributions are applied to calculate the static wind effect independently of a linear or possible non-linear behaviour of the structure. If a linear structural behaviour can be assumed the quasi-static component of the wind load is a linear function of the variances and co-variances of the pressure fluctuations. The resonant load component can be expressed by the spectral density and the cross spectral density function in the small band widths of dominating natural frequencies. Such procedure permits to evaluate equivalent static forces which fully include the quasi-static and resonant gust effect.

3.2 Wind forces and wind pressures

3.2.1 Parameters

Three-dimensional flow field:

The pressures vary over the height of the tower for three reasons:

- The undisturbed flow performs in parallel layers of different velocities. At the surface of the tower developes a vertical pressure gradient which diverts a part of the flow lines and generates a three-dimensional flow.

- The flow over the upper end of the tower refills the low pressure at the leeward side of the tower which reduces total drag. Such tip effect is not present in the middle range of the shaft. Its total influence is expressed by a reduction factor and depends on the aerodynamic slenderness of the structure. The aerodynamic slenderness of the tower is h/D = 1000/130 = 7,69.

- Two conical vortices develop at the upper end of the tower and locally increase the drag.

Reynolds number: Re is 4.10^8 and trans-critical which means that the aerodynamic coefficients are independent of Re.

Surface roughness: The pressure distribution and the drag forces are influenced by the surface rougness of the structure even if Re is trans-critical. Achenbach and Heinecke specify $c_f = 0,6$ for a smooth surface and $c_f = 1,0$ for a rough surface, both values are valid in case of a large aerodynamic slenderness. A large roughness increases the total drag and reduces maximum suction. The latter is a favourable effect regarding the local behaviour of the tower shell. The following roughness values can be expected:

Table 3.1: Roughness of the surface of the tower

board-marked concrete surface	k = 3mm	k/D = 0.003/130 = $2.3\ 10^{-5}$
roughness due to cranks	k = 13mm	k/D = 10^{-4}

For this study a smooth surface is assumed.

3.2.2 Wind forces

Wind forces result from the integration of the wind pressures and are applied to calculate global reaction forces as displacements and internal beam forces.

The concept of the enveloping gust can be applied in order to evaluate gust effects if the resonant effects remain small. In this study, the gust response is calculated from the mean, static reaction to wind which is multiplied by a gust response factor, G > 1,0.

A distributed wind force and the respective force coefficient can be defined by:

$$F_W(z) = c_f(z) \cdot D \cdot q(z), \quad c_f(z) = \frac{F_W(z)}{D \cdot q(z)} \qquad (3.1a,b)$$

The influence of the profile of the dynamic pressure over height is separated from other effects of the three-dimensional flow over the structure in the equations above. It is a common simplification to assume the force coefficient c_f to be constant over height:

Table 3.2: Drag force coefficients at the shaft apart from the tip

DIN 1056	concrete chimneys, fixed surface roughness, slenderness variable	$c_f = c_{f0}\ \psi$ = 0.95 (0.65 + 0.005 h/D) = 0.69
expert's report [2]	slenderness and surface rougness variable	$c_f = 0.52$ for k/D = $1.5\ 10^{-6}$
expert's report [3]	no further specifications	$c_f = 0.5$ +/- 0.1
after Ruscheweyh [4]	from pressure measurements at the telecommunication tower of Hamburg	$c_f = 0.49$ for k/D = $3.7\ 10^{-4}$ bis $1.1\ 10^{-3}$; h/D > 11; Re = $1.4\ 10^7$;
DIN 1055-4: 2005-03	extrapolation for Re = $5 \cdot 10^8$ as the indicated range of Re end at $5 \cdot 10^7$. reduction factor due to slenderness: ψ = 0.67	$c_f = 0.67$ x 0.9 = 0.60

The fourth value is can not be compared to the others as it is evaluated from extreme peaks of the dynamic pressures and resulting surface pressures. The other values are evaluated as time mean values in turbulent flows. The force coefficient c_{f0} = 0,6 is the adequate choice regarding the assumed surface roughness and the aerodynamic slenderness of the tower. The coefficient's variability with height in Eq. (3.1a, b) is modelled by

$$c_f(z) = c_{f0}\left(\frac{z}{h}\right)^{-0,4\alpha_q} \quad z > 50\text{ m} \tag{3.2}$$

in order to increase the accuracy of the calculation.

An additional load ΔF_W covers the increased wind load at the tip of the tower.

$$\Delta F_W = 0,2 \cdot D \cdot q(h) \quad \text{for } (h\text{-}2D) < z < h \tag{3.3}$$

3.2.3 Wind pressures

The stresses of the tower shell depend on the magnitude and the distribution of the wind pressures. The pressure fluctuations are generated by gusts and by separation-induced turbulence and contain a broad band of frequencies. Most of the energy lies apart from the range of eigenfrequencies of the structure. Therefore, the quasi-static reaction is prevailing.

The mean and the fluctuating pressures together with the correlations are required to calculate the static and quasi-static parts of the structural responses with a sufficient accuracy for structural design.

Mean coefficients for the pressures on the external surface

The aerodynamic pressure coefficient is defined by

$$c_p(z,\varphi) = \frac{p_m(z,\varphi)}{q_m(z)} \tag{3.4}$$

(z – height above ground, φ - angle of circumference in cylindrical coordinates, q – dynamic pressure).

The enormous height of the tower requires to modelling the variability of the pressure coefficients with height. The following reference values according to [7] are used in order to calibrate the pressure profile:

- coefficient of base pressure:

$$c_{pb} = -0,45\left(\frac{z}{h}\right)^{-0.8\alpha_q} \tag{3.5}$$

- maximum pressure along the stagnation line

$$\max c_p = 1,0 \tag{3.6}$$

Both values are not affected by the surface roughness. The increase of pressure from the pressure minimum min c_p in downstream direction strongly depends on the surface roughness while it can be assumed to be independent of $k/D = 10^{-4}$.

$$\Delta c_p = 1,0 \text{ and } \min c_p = c_{pb} - \Delta c_p \tag{3.7a,b}$$

The circumferential distribution of the pressure coefficients is derived with reference to empirical formulas from wind tunnel measurements and measurements in natural scale at cooling towers [7].

- range I: decrease of pressure from the maximum at the stagnation line at $\varphi = 0$ towards the pressure minimum at $\varphi = \varphi_1$ (3.8a,b)

$$\varphi_1 = 50 + (\max c_p - \min c_p)10$$

$$c_p = \max c_p - (\max c_p - \min c_p)\left[\sin\left(\frac{90}{\varphi_1}\varphi\right)\right]^{2,4}$$

- range II: increase of pressure from the minimum at $\varphi = \varphi_1$ towards the boundary of the wake at φ_N

$$\varphi_N = 60 + (0,46 - c_{pb})(18 + 33 \cdot \Delta c_p)$$

$$c_p = \min c_p + \Delta c_p\left[\sin\left(\frac{\varphi - \varphi_1}{\varphi_N - \varphi_1}90\right)\right]^{2,4} \tag{3.9a,b}$$

- range III: constant pressure in the wake between $\varphi_N < \varphi < 180°$:

$$c_p = \text{const.} = c_{pb} \tag{3.10}$$

The distribution of pressures is symmetrical at $\varphi > 180°$.

Coefficient of the fluctuations of external pressures and r.m.s. value σ_p

Circumferential distribution normalized by the values at the stagnation line:

$$\frac{\sigma_p(\varphi,z)}{\sigma_p(\varphi=0,z)} = 1,0 \qquad 0 < \varphi < \varphi_N$$

$$= 0,5 \qquad \varphi_N < \varphi < 180° \tag{3.11a,b}$$

Distribution over height at the stagnation line:

$$\frac{\sigma_p(z,\varphi=0)}{\max p(z)} = \frac{\sigma_p}{q(z)} = 1,8 \cdot I_u(z) \tag{3.12}$$

The fluctuations of the external pressures are correlated in vertical and circumferential directions. Further specifications for more detailed investigations can be derived from available, numerous experimental results.

Coefficients for internal suction

The internal pressure is dominated by the flow characteristics over the tip of the tower. The mean pressure is approximately constant over the height and the internal circumference of the tower.

$$p_{mi} = -0{,}45 \cdot q_m(h) \tag{3.13}$$

The r.m.s. value of the fluctuation of the internal pressures can also be assumed to be constant over the height and the internal circumference with the exception of a certain range at the upper end of the tower shaft.

$$\sigma_{pi} = 1{,}0 \cdot I_{uh} \cdot p_{mi} \quad z<h–D/2 \tag{3.14}$$

The flow over the structure can under certain conditions enter into the tower which will form a inhomogeneous distribution of the r.m.s. value which is modelled by:

$$\begin{aligned} \sigma_{pi}(\varphi = 0) &= 1{,}0 \cdot I_{uh} \cdot p_{mi} \\ \sigma_{pi}(\varphi = 180°) &= 2{,}0 \cdot I_{uh} \cdot p_{mi} \end{aligned} \quad z>h\text{-}D/2 \tag{3.15}$$

The fluctuations of the internal pressure are fully correlated, the correlation to the external pressures is zero.

3.2.4 *Equivalent static loads for the quasi-static gust effect*

The instantaneous upper and lower peak values of the pressures p_p are relevant for the structural design and are modelled by:

$$p_p = c_p q_m \pm g \cdot \sigma_p \tag{3.16}$$

The positive or negative sign of the second component follows the sign of the mean part. The peak factor g depends on the extent of the area which influences the load.

The shell is susceptible with respect to membrane tension in meridional directions for which the peak values of the pressure on the windward side and of the suction on the flanks are relevant. With $c_p = \max c_p$ and $\sigma_p = 1{,}8\ I_u \cdot \max p_m$ is the maximum pressure and analogously the minimum pressure:

$$\begin{aligned} \max p_p &= \max p_m (1 + g \cdot 1{,}8 \cdot I_u) \\ \min p_p &= q_m \cdot \min c_p - g \cdot 1{,}8 \cdot I_u \cdot \max c_p \cdot q_m \\ &= \min p_m \left(1 - g \cdot 1{,}8 \cdot I_u \frac{\max c_p}{\min c_p}\right) \end{aligned}$$

(3.17 a,b,c)

The absolute value of $\min c_p$ is larger than the absolute value $\max c_p$ in the considered case. The amplification factor of the windward side is used. In a conservative approach the equivalent load is:

$$\begin{aligned} &p(z,\varphi) = \\ &= c_p(z,\varphi) \cdot q_m(z)\left(1 + g \cdot 1{,}8 \cdot I_u(z)\right) \end{aligned} \tag{3.18}$$

The increased mean pressure can be understood as a gust pressure q_b. The peak factor is assumed to be $g = 3{,}0$ due to the quasi-static behaviour and to the larger influence areas. The exponent $\alpha_q = 0{,}3$ is used for the profile of the mean dynamic pressure.

$$\begin{aligned} q_b(z) &= q_{m10}\left(\frac{z}{10}\right)^{0{,}30}\left(1 + 3{,}0 \cdot 1{,}8 \cdot I_u(z)\right) \\ &\approx 2{,}24 \cdot q_{m10}\left(\frac{z}{10}\right)^{0{,}18} \end{aligned} \tag{3.19}$$

3.3 *Vortex excitation*

Wind gusts, separation-induced turbulence including quasi-periodic vortex separations generate the pressure fluctuations. The measured pressure fluctuations contain the quasi-static effect of the vortex separation.

The contribution of vortex-resonance is limited to specific frequencies and can be the origin of ovalizing oscillations. It is proofed in a detailed investigation in [7] that vortex resonance must not be expected as the wind velocity at the assumed building locations will not arrive at critical values.

Table 3.3: Lower limit velocity for the occurence of lock-in

	V_{crit}	lower limit for lock-in	existing V
top range	90.0 m/s	79.2 m/s	52.0 m/s
shaft range	58.5 m/s	55.6 m/s	50.4 m/s

4 WIND RESPONSE FOR THE BEAM-LIKE BEHAVIOUR OF THE TOWER SHAFT

The gust response factor for the base bending moment due to longitudinal gusts is analysed with the following parameters:

Table 4.1: Parameters of the building and of the design wind at the building location after AS 11 70.2 -1989

Building height	$h = 1000$ m
Diameter	$b = 130$ m
Air density	$\rho = 1{,}2$ kg/m³
Mean wind speed (1-h mean) in building height	$\overline{v}_h = 60$ m/s
Profile exponent	$\alpha_v = 0{,}15$; $\alpha_q = 0{,}29$

Integral length scale	L_h = 3160 m
Eigenfrequency of the 1st bending mode	n_1 = 0,09 Hz
Background noise	B = 0,342 (w = 0,0562)
Damping decrement	δ=0,08

The computational results of an analysis after the Australian code are compared to respective results after the Eurocode:

Table 4.2: Compendium for the computation of the gust response factor

	SAA	Eurocode
Quasi-static contribution	$B(1+w)^2 = 0.382$	$Q_0^2 = 0.350$
Spectral value	E = 0.155	R_N = 0.155
Size factor	S = 0.090	R_b R_h = 0.074
Resonant contribution	$\frac{S\cdot E}{\varsigma} = \frac{2\pi S\cdot E}{\delta} = \frac{0.0877}{\delta}$	$\frac{\pi^2}{2\delta} R_N R_h R_b = \frac{0.0566}{\delta}$
Peak factor, quasi-static Peak factor, resonant	g_v = 3.7 g_f = 3.4	g = 3.04 uniform for both parts
turbulence intensity $I_u(z_{ref})$	z_{ref} = h I_u = 0.052	z_{ref} = 0.6 h I_u = 0.069
Gust response factor G	1.440	1.431
Resonance factor G/G_Q	1.163	1.156

The comparison shows that both procedures provide similar values for the quasi-static response. The peak factor after the Eurocode is smaller because the peak value of the response refers to the 10-min mean whereas the SAA code uses the 1-h mean. Differences of the applied wind spectra lead to smaller resonance components after the Eurocode. However, this has no dominant influence on complete result.

Both codes generate the same gust response factor for the base bending moment. The resulting equivalent static wind load for the base bending moment after the Eurocode is 10% larger than the same quantity calculated after the Australian code, since the Eurocode refers to the higher 10-min mean instead of the 1-h mean in SAA.

5 CONCLUSIONS

There is a lack of wind engineering data for heights of larger than 300 m above ground. The required properties of the turbulent wind in such a height are derived from the theories of the Prandtl and the Ekman layers as engineering models. This is especially important for the turbulence structure which controls the dynamic wind forces. The wind load in the case of a tower with a height of 1000 m and more can not be introduced as usual by a quasi-static gust wind load. Rather, it must be separated into its mean and fluctuating components.

On the one hand the structural behaviour is that of a shell exposed to inhomogeneous, mean and fluctuating pressure distributions. The distribution of the mean pressures depends on the very high Reynolds number and the wind profile. The fluctuating pressures are dominated by the turbulence intensity and the separation of the mean flow. They can be represented by their variances and co-variances or, alternatively in the time domain. Resonant contributions are not relevant. The experience with large cooling towers provides important input to these questions. On the other hand the shaft responds like a beam under correlated, longitudinal and lateral forces. The reactions are of mean, quasi-static and resonant type. The concept of the gust response factor provides a suitable tool for the calculation of the related response quantities due to a longitudinal gust impact. Calculational concepts for the lateral responses are available, e.g. [5].

A set of wind loading data is presented which makes it possible to design these outstanding structures and perform important statical proofs.

6 REFERENCES

[1] Schlaich Bergermann Solar GmbH: The solar updraft tower. ISBN 3-934369-51-0, Bauwerk Verlag, Berlin 2004

[2] Wacker Ingenieure: Solar Chimney Project: Investigation of wind loads and related aspects by means of wind tunnel tests and theoretical approaches. Internal report, April 2001

[3] A.G. Davenport: The Solar Chimney: The Action of the Wind. Internal report

[4] H. Ruscheweyh: Beitrag zur Windbelastung hoher kreiszylinderähnlicher schlanker Bauwerke im natürlichen Wind bei Reynoldszahlen bis Re=1,4x10^7. Dissertation RWTH Aachen 1974

[5] ESDU: Response of structures to vortex shedding – structures of circular or polygonal cross-section. Item No 96030, London 1996

[6] Harris, R.I., & D.M. Deaves: The Structure of Strong Winds. Proceedings CIRIA Conf., London 1980

[7] Niemann, H.-J.: Windwirkungen auf hohe Schalenkühltürme. In: Bautechnik in Wärmekraftwerken, S. 248 – 269. Verlag VGB-Kraftwerkstechnik, Essen 1993

[8] Australian code AS 11 70.2 – 1989: SAA Loading Code Part 2: Wind Loads