Quantitative Relationship of Wind Vibration Coefficient, Average Wind Speed and Damping Coefficient of United Framework of Transformer Substation

Xue-jun TANG¹, Dong XIE¹*, Yi XIONG¹, Hong-chuan DONG¹, Zhao-yang ZHANG¹, Lu-ping LI², Feng-hua JIN², Jiang FENG², Shang-jun YAN², Xiang-min CHEN², Hao ZHANG² and Heng DING¹

¹State Grid Hubei Economic Technology Research Institute, Wuhan, 430077, China
²Changsha University of Science and Technology, Changsha, 410014, China
*Corresponding author

Keywords: Transformer substation, Full united framework, Wind vibration coefficient, Average wind speed, Damping coefficient, Finite element analysis.

Abstract. The united frameworks of UHV and EHV substation belong to wind-sensitive structures, and wind load is one of the main control loads of the design. In this paper, a 500kV full united substation framework was used as the engineering background to explore the quantitative relationship between wind-induced vibration coefficient, damping coefficient and average wind speed of the full united framework. The results showed that the average wind speed and the damping coefficient of the framework have significant influence on the wind vibration coefficient of the framework. The research results in this paper not only provide theoretical and computational basis for the wind resistant design of the combined framework structures, but also provide a basis for formulating the design criteria of full united wind vibration coefficient.

Introduction

With the transformation of China's power transmission mode toward ultra high voltage (UHV) and extra high voltage (EHV) transmission technology, 500kV and 1000kV substation frameworks become one of the most important links in power grid system. Moreover, the cost of the frameworks occupies a large proportion in the entire transmission and distribution lines, and their security directly affects the safety and reliability of the entire power grid. In order to ensure the safety and reduce the project cost as far as possible, the most commonly used method is to use the full united framework layout. Different from the conventional substation frame, the load and its height and span of the 500kV and 1000kV fully united substation frame have a greater degree of increase, which have the characteristics of high height, heavy load and low self vibration frequency compared with other lower voltage grades [1]. In addition, the whole united transformer framework is provided with a frame beam at different height points according to the need of structural layout. This makes the windward side of the structure larger than that of the single portal type structure, and meanwhile, the whole structure tends to be flexible, making effect of wind load significant. Therefore, the structure is wind sensitive, and wind load is often the main or even decisive control load of design.

In this study, a 500kV full united substation framework around the middle reaches of the Yangtze River was used as an engineering background. Based on random vibration theory, the wind vibration response and wind vibration coefficient of the transformer framework under different average wind speed and fluctuating wind speed were calculated and analyzed. The aim was to explore an effective way to obtain the wind load response characteristics of the designed structure quickly in the design stage, providing a reference for the optimization design of wind resistant structure and defect diagnosis.
Calculation Object and Calculation Model

Brief Introduction to Calculation Object

The 3D solid model of the calculated united framework is shown in Fig. 1. Cross braces of the united framework were set up at the height of 12m and 23.5m, with chord member being a steel tube, and web member being a corner steel member. Unstiffened flange connection was adopted for beam pipe joint, while stiffened flange connection was adopted for steel pipe chord. Steel tubular chord, angle steel web and bolt connection formed the lattice steel beam. The joint frame was equipped with a frame beam at the height of 28m. The beams of the joint frame shown in Fig. 1 are numbered 1 to 7 from left to right. The definition of the direction of the incoming line of the substation framework, the direction angle of the wind and coordinate system are shown in Fig. 2.

![Figure 1. 3D solid model of 500kV full united framework.](image)

![Figure 2. Incoming line of substation framework, coordinate direction and wind direction.](image)

Calculation Model of Wind Vibration Coefficient

Displacement wind vibration coefficient is one of the most commonly used wind vibration coefficients. The displacement response includes the response components of mean load and fluctuating load. In this study, the displacement wind vibration coefficient is used for calculation. According to the regulation of building structure load, the formula of wind vibration coefficient is calculated as [2].

\[
\beta = \frac{\mu_u + g\sigma_u}{\mu_u} \quad (1)
\]

where
- \( \beta \) is the wind vibration coefficient without dimension;
- \( \mu_u \) is the displacement of average wind load, \( m \);
- \( \sigma_u \) is the displacement with variance under fluctuating wind load, \( m \);
where

\[ g = \sqrt{2\ln(v_0T) + \frac{0.5772}{\sqrt{2\ln(v_0T)}}} \]  

(3)

where

- \( T \) is the observation time of one hour, \( h \);
- \( v_0 \) is the zero crossing rate in unit time, \( \frac{1}{h} \);
- \( v_0 = \frac{1}{2\pi} \sqrt{\frac{2}{\lambda_0}} \), spectral moment;
- \( \lambda_0 = 2\int_0^\infty n'G(n)\,dn \)

Wind Load Calculation Model

The force acting on a united framework in fluctuating wind loads can also be expressed in terms of spectrum, called wind spectrum. The wind is composed of the steady part and the pulsating part, and the steady-state part is larger than the pulsating part.

**Calculation of standard wind pressure.** When the wind moves forward at a certain speed, it will create pressure on the obstruction, called wind pressure. The calculation model of wind pressure is [3]:

\[ w = \frac{\rho V^2}{2} = \frac{\gamma V^2}{2g} \]  

(4)

where

- \( w \) is wind pressure, \( kN/m^2 \) (Pa);
- \( V \) is wind speed, \( m/s \);
- \( \gamma \) is air unit weight, \( m^3/kg \);
- \( \rho \) is air density, \( kN/m^3 \);
- \( g \) is gravitational acceleration, \( m/s^2 \).

Eq. (4) is standard wind pressure calculation formula. Under the standard condition (air pressure is 1013 \( hPa \), and the temperature is 15 degrees Celsius), the air unit weight is \( \gamma = 0.01225 \ kN/m^3 \). Gravity acceleration at 45 degrees latitude is \( g = 9.8\ m/s^2 \), and the following can be derived [4].

\[ w_0 = \frac{1}{1600} V^2 \]  

(5)

**General formula for calculating wind pressure.** In engineering practice, since the average wind speed varies with the change of elevation and the pulsation of wind speed, the general wind pressure formula is [5].
\[ w(z,t) = \frac{2\mu}{2g} \left[ \bar{V}(z) + V(z,t) \right]^2 \]  

(6)

where  
\[ \bar{V}(z), V(z,t) \] represent one hour mean velocity part and pulsating velocity part at height \( z \), \( m/s \).

The variation of average wind speed along the height is calculated by the above exponential model; \( \mu \) is the shape coefficient of the structure.

The variation law of the average wind speed along the height can be approximately described by exponential function [3].

\[ \bar{V}(z) = \left( \frac{z}{z_s} \right)^\alpha \]  

(7)

where \( \bar{V}(z) \) is the average wind speed at height \( z \), \( m/s \);

\( \bar{V}(z_s) \) is the average wind speed at reference height, \( m/s \). Generally, the reference height is set to \( z_s = 10m \), and \( \bar{V}(z_s) \) is set to \( 30 m/s \) for calculation;

\( \alpha \) is surface roughness index. According to the regulation in [3], the studied object locates in B class region, and \( \alpha = 0.16 \).

**Wind force calculation.** Generally, wind loads acting on structures can be expressed in the following formula [6].

\[ F_w = w(z,t)A = \frac{2\mu}{2g} A \left[ \bar{V}(z) + V(z,t) \right]^2 \]  

(8)

where \( A \) is the structure area under wind pressure, \( m^2 \). We have [5]:

\[ \left[ \bar{V}(z) + V(z,t) \right]^2 = \bar{V}^2(z) \left[ 1 + 2 \frac{V(z,t)}{\bar{V}(z)} + \frac{V^2(z,t)}{\bar{V}(z)} \right] \]  

(9)

Therefore, the wind load can be expressed as:

\[ F_w = F_w^{(1)} + F_v + F_{v^2} \]  

(10)

It is clear in (10) that the wind load acting on the framework is made up of three parts. Since the fluctuating wind speed is much smaller than the average wind speed, the square term of fluctuating wind speed can be neglected when calculating wind load. Then,

\[ F_w \approx F_w^{(1)} + F_v \]  

(11)

(1) steady state force \( F_w^{(1)} \)

\[ F_v = \frac{2\mu}{2g} A \bar{V}^2 \]  

(12)

This is the usual sense of wind load, but this is based on the average wind speed of 1 hours, and the increase of load caused by gust is another consideration.

(2) Synchronous pulsating wind force \( F_v \)

\[ F_v = \frac{2\mu}{2g} A \bar{V}^2 \left( \frac{2}{\bar{V}} \right) V(t) = F_w^{(1)} \frac{2}{\bar{V}} V(t) \]  

(13)

Therefore, there is only a constant factor between the wind spectrum of \( S_{w_i}^{(1)}(\omega) \) and the \( S_v(\omega) \) [7]:
Calculation Model of Pulsating Response of Substation Framework

In this study, wind vibration response theory based on frequency domain method was applied to calculate the wind vibration response characteristics of the united substation framework structure. Based on the modal decomposition method, the frequency domain method uses stochastic vibration theory to solve the dynamic wind-induced responses of structures under fluctuating wind loads.

Basic hypothesis. Based on the frequency domain method, the following assumptions are applied to the calculation of wind-induced response. (1) The effect of fluctuating wind turbulence on the substation framework is a stationary random process; (2) Under the action of wind load, the substation framework is in a linear elastic state without the need of considering the nonlinear effect of the substation framework under fluctuating wind load; (3) Quasi constant hypothesis (the wind pressure on the surface of the building structure fluctuates synchronously with the incoming wind speed and pressure, without considering the influence of pneumatic admittance), i.e., ignoring the coupling effect of the fluctuating wind and the substation framework.

Motion equation of substation framework and its solution. The structural vibration equation induced by fluctuating wind load $F_v(t)$ on a substation framework can be expressed as[3]

$$M \ddot{x}(t) + C \dot{x}(t) + K x(t) = F_v(t)$$

(15)

where
- $M$ is the $n$-order mass matrix;
- $C$ is the $n$-order damping matrix;
- $K$ is the $n$-order stiffness matrix;
- $x(t)$, $\dot{x}(t)$, and $\ddot{x}(t)$ are the displacement, velocity and acceleration vectors, respectively;
- $F_v(t)$ is node pulsating wind load vector.

Use modal method to decouple for (15), and let:

$$x(t) = [\phi] y(t)$$

(16)

where $[\phi] = [\phi_1, \phi_2, \phi_3, \ldots, \phi_n]$ is mode vector.

The pulsating wind load of the node is [9]:

$$F_i(t) = \rho C_{Li} V y(t) A_i$$

(17)

where
- $\rho$ is air density;
- $A_i$ is the sum of all the projection areas of the steel pipe on the windward face at framework node $i$;
- $C_{Li}$ is the average wind pressure coefficient at node $i$.

For the small damping system of the substation framework, the spectral density function of the displacement vector $x(t)$ under the action of wind load is[10]

$$S_x(2\pi n) = \sum_{j=1}^{n} S_{\phi_j}((2\pi n) t)$$

(18)

where
- $\phi_j$ is the vibration vector of type $j$;
- $S_{\phi_j}(2\pi n)$ is the self spectral density function of generalized force $S_{\phi_j}(t)$;
\[ |H_j(2\pi in)| \] is the module of the frequency response function of generalized force \( S_{F_j}(t) \).

The autocorrelation function of generalized force \( S_{F_j}(t) \) is [11]:

\[
S_{F_j}(\tau) = \sum_{r=1}^{n} \sum_{s=1}^{n} \phi_j \phi_d R_{F_j}(r,s,\tau) \tag{19}
\]

Conduct Fourier transform for the above equation to obtain the self spectral density of generalized force:

\[
S_{F_j}(2n\pi) = \sum_{r=1}^{n} \sum_{s=1}^{n} \phi_j \phi_d S_{F_j}(r,s,n) \tag{20}
\]

The expression of its cross spectral density is [2]:

\[
S_{F_jF_k}(2n\pi) = \frac{1}{2\pi} \int_{-\pi}^{\pi} R_{F_jF_k}(\tau) e^{-2\pi in\tau} d\tau = \rho^2 C_{L_r} C_{L_s} A_r A_s V_r V_s S_{nn}(2n\pi) \tag{21}
\]

where \( C_{L_r}, C_{L_s} \) are the average wind pressure coefficients of \( r,s \), respectively, without dimension;
\( V_r, V_s \) are the average wind speed of \( r,s \), \( m/s \);
\( S_{nn}(2n\pi) \) is the pulse wind speed cross spectral density between \( r,s \).

**Relationship between Maximum Pulsation Response Displacement and Average Wind Speed of United Framework**

Under the effect of 0° wind direction load, the maximum pulsation response value of the substation framework versus the change of average wind speed is shown in Fig. 3(a). The quantitative relation between the maximum pulsation response displacement and the average wind speed satisfies the following:

\[
y(\bar{V}) = 5.8 \times 10^{-3} \bar{V}^3 - 0.4 \times 10^{-2} \bar{V}^2 + 0.12 \bar{V} - 0.9286 \tag{22}
\]

Under the effect of 90° wind direction load, the maximum pulsation response value of the substation framework versus the change of average wind speed is shown in Fig. 3(b). The quantitative relation between the maximum pulsation response displacement and the average wind speed satisfies the following:

\[
y(\bar{V}) = 8.67 \times 10^{-5} \bar{V}^3 - 7.25 \times 10^{-3} \bar{V}^2 + 0.21 \bar{V} - 1.759 \tag{23}
\]

Under the effect of 45° wind direction load, the maximum pulsation response value of the substation framework versus the change of average wind speed is shown in Fig. 3(c). The quantitative relation between the maximum pulsation response displacement in \( x \) and \( y \) direction and the average wind speed satisfies the following:

\[
x \text{ direction: } y(\bar{V}) = 1.2 \times 10^{-5} \bar{V}^3 - 1 \times 10^{-3} \bar{V}^2 + 0.0293 \bar{V} - 0.2388 \tag{24}
\]

\[
y \text{ direction: } y(\bar{V}) = 1.733 \times 10^{-5} \bar{V}^3 - 1.45 \times 10^{-3} \bar{V}^2 + 4.245 \times 10^{-2} \bar{V} - 0.3487 \tag{25}
\]
Figure 3. Relationship between maximum pulsation response of substation framework and average wind speed at reference height.

**Relationship between Wind Vibration Coefficient and Average Wind Speed of United Framework**

Under the effect of 0° wind direction load, the relationship between the wind vibration coefficient of the substation framework and the average wind speed under different average wind speed is shown in Fig. 4(a). The quantitative relation between the wind vibration coefficient and the average wind speed satisfies the following:

$$\beta_i(\bar{v}) = 1.0667 \times 10^{-4} \bar{v}^3 - 7.51 \times 10^{-3} \bar{v}^2 + 0.182\bar{v} + 0.48829$$  \hspace{1cm} (26)$$

Under the effect of 90° wind direction load, the relationship between the wind vibration coefficient of the substation framework and the average wind speed under different average wind speed is shown in Fig. 4(b). The quantitative relation between the wind vibration coefficient and the average wind speed satisfies the following:

$$\beta_i(\bar{v}) = 4.0 \times 10^{-5} \bar{v}^3 - 2.37 \times 10^{-3} \bar{v}^2 + 6.657 \times 10^{-2} \bar{v} + 1.4894$$  \hspace{1cm} (27)$$

Under the effect of 45° wind direction load, the relationship between the wind vibration coefficient of the substation framework and the average wind speed under different average wind speed is shown in Fig. 4(c). The quantitative relation between the wind vibration coefficient and the average wind speed satisfies the following:

- **x direction:**  \( \beta_i(\bar{v}) = 6.6667 \times 10^{-5} \bar{v}^3 - 4.46 \times 10^{-3} \bar{v}^2 + 0.1132 \times 10^{-2} \bar{v} + 1.4391 \)  \hspace{1cm} (28)$$

- **y direction:**  \( \beta_i(\bar{v}) = -2.6667 \times 10^{-5} \bar{v}^3 + 1.71 \times 10^{-3} \bar{v}^2 - 1.505 \times 10^{-2} \bar{v} + 2.4097 \)  \hspace{1cm} (29)$$
Relationship between Wind Vibration Coefficient and Damping Coefficient of United Framework

Under the wind load of 0° wind direction, the relationship between the wind vibration coefficient of the substation framework and the damping ratio is shown in Fig. 5(a). The quantitative relation between the wind vibration coefficient and the damping ratio satisfies the following:

$$\beta_i(\xi) = -4000\xi^2 + 118\xi + 1.28$$  \hspace{1cm} (30)$$

Under the wind load of 90° wind direction, the relationship between the wind vibration coefficient of the substation framework and the damping ratio is shown in Fig. 5(b). The quantitative relation between the wind vibration coefficient and the damping ratio satisfies the following:

$$\beta_i(\xi) = -750\xi^2 + 9.5\xi + 2.41$$  \hspace{1cm} (31)$$

Under the wind load of 45° wind direction, the relationship between the wind vibration coefficient of the substation framework and the damping ratio is shown in Fig. 5(c). The quantitative relation between the wind vibration coefficient and the damping ratio satisfies the following:

For the \( x \) direction:

$$\beta_i(\xi) = -150\xi^2 - 8.5\xi + 2.72$$  \hspace{1cm} (32)$$

For the \( y \) direction:

$$\beta_i(\xi) = 400\xi^2 - 38\xi + 3.12$$  \hspace{1cm} (33)$$
Comparisons indicate that the average wind speed has a significant influence on the maximum pulsation response displacement of the whole united framework, and the maximum fluctuation response displacement of the node increases significantly with the increase of the average wind speed. The relationship between average wind speed and maximum pulsation response of the united framework is basically power three, and in a partial interval of average wind speed, it can be regarded as an approximate linear relationship.

It is clear in Figs. 4 that the average wind speed has a greater influence on the wind vibration coefficient of the united framework. The node's wind vibration coefficient increases with the increase of average wind speed. The relationship between average wind speed and the wind vibration coefficient of the united framework is basically power three, but in a relatively large average wind speed interval (15~30 m/s), it can be regarded as an approximate linear relationship.

It is clear in Figs. 5 that the structural damping ratio has a certain effect on the wind vibration coefficient of the united framework. The wind vibration coefficient decreases with the increase of damping ratio, and vice versa. The relationship between the structural damping ratio and the wind vibration coefficient of the whole united framework is basically power two, and can also be approximated as a linear relationship.

**Acknowledgement**

This research was financially supported by the National Science Foundation. Science and technology project of Hubei electric power company of State Grid (Testing technology and engineering application of damping ratio and wind vibration coefficient of full united substation framework, 52153816001B)
References


