EQUIVALENT STATIC BUFFETING LOADS ON STRUCTURES

By Yin Zhou, Ahsan Kareem, and Ming Gu

ABSTRACT: The loads on buildings and structures caused by the buffeting action of wind have traditionally been analyzed using the gust loading factor (GLF) approach in most codes and standards. In this approach, the equivalent static wind loading used in design is equal to the mean wind force multiplied by the GLF. Although the traditional GLF method can ensure an accurate estimation of the displacement response, it may not provide a reliable estimate of other response components. In addition, this method fails to provide any guidance in cases with a zero mean response. This note presents a theoretical formulation for the buffeting wind loading on structures, which eliminates these shortcomings. A tall building and a long-span bridge are used to demonstrate the effectiveness and improved predictive features of the proposed formulation.

INTRODUCTION

The original method for determining the dynamic response and equivalent wind loads due to the buffeting action of wind on structures such as tall buildings, towers, and bridges was proposed by Davenport (1967). This method led to the concept of the “gust loading factor” (GLF). Davenport’s initial formulation has been further advanced in a number of studies, including Vickery (1970), Simiu (1980), Solari (1993a,b), Solari and Kareem (1998), and others.

In the GLF method, the equivalent wind load on structures is treated as the mean wind force multiplied by the GLF as follows:

\[ \tilde{P}(z) = G\tilde{P}(z) \]

where the constant \( G \) is formulated based on the displacement response and equivalent static wind loads, respectively.

Because of its simplicity and generality, the GLF method has been used in most wind loading codes and standards around the world. Despite this universal acceptance, Zhou et al. (1999a, b) noted that the equivalent static wind load on tall buildings set out with the GLF method, which follows the distribution of the mean wind force, differs significantly from the inertial wind loading that is related to the mass and mode shape. Because of this difference in wind loading, significantly inconsistent results have been found in the estimation of some response components based on the GLF method. This shortcoming in the definition of wind loading with the GLF method does not exist only for tall buildings. In addition, the shortcoming is present for other flexible and lightly damped structures for which the resonant response is dominant.

More commonly understood issue relates to the inability of the GLF-based approach to provide a prediction for structures with a zero mean response, such as a suspension bridge or a cable-stayed bridge with an asymmetrical mode shape. In such a situation, the GLF method does not provide any meaningful guidance.

This note introduces a theoretical formulation for the equivalent static buffeting wind loading. The overall equivalent static wind loading is divided into its mean, background, and resonant components. The resonant component is expressed in terms of the inertial load, and the background component is derived based on the “L.R.C. method” (Kasperski and Niemann 1992). The proposed approach applies to both vertical and horizontal structures, including those with zero mean displacement.

WIND LOADS ON VERTICAL STRUCTURES

A uniform rectangular tall building with height \( H \), width \( B \), and depth \( D \) is considered here. The structural features are as follows: mass per unit height \( m(z) = m_1 \) first mode shape \( \psi(z) = (z/H)\beta \); \( \beta \) is constant; and the natural frequency and damping ratio in the first mode are \( f_1 \) and \( \zeta_1 \), respectively. The equivalent static wind load is expressed in terms of the mean, background, and resonant components. For the unsteady components, this formulation is based on statistical integration of the random pressure field around the building surface (Kareem 1982).

The mean wind force is given by the following:

\[ P(z) = \frac{1}{2} \rho U_H^2 C_d(z/H) \alpha \]

where \( U_H \) = mean wind velocity at the top height of the building; \( \rho \) = air density; \( \alpha \) = mean wind velocity profile exponent; and \( C_d \) = drag force coefficient. This expression can be recast to account for loading on the windward and leeward faces as used in ASCE 7-95.

The determination of the equivalent static wind load associated with the background response can be best described in terms of the correlation of the random pressure field and appropriate load effect, which is defined using an appropriate influence function as given by Kasperski and Niemann (1992). If the correlation coefficient between the fluctuating wind pressure at height \( z \) and the associated load effect at height \( z' \) can be denoted by \( Q(z) \), then the peak background load can be given by

\[ \tilde{P}_B(z) = g_s Q(z) \sigma_s(z) \]

where \( g_s \) = background peak factor (Zhou and Kareem, manuscripts).
Equivalent static wind loads on horizontal structures can be derived using the above procedure given for vertical structures. For these structures, the expressions are generally much simpler, because it usually can be assumed that the wind speed does not vary along the span. Using the buffeting theory of Scanlan and Gade (1977), expressions for equivalent buffeting wind loads in the vertical mode on horizontal decks are summarized in the following. This method can easily be extended to other horizontal structures and also used for qualifying torsional load effects.

The mean wind loading is given by

\[ P_{x}(x) = \frac{1}{2} \rho C_{x} \alpha_{0} B \]  

where \( P_{x}(x) \) = mean wind load in the vertical direction at \( x \); \( \rho \) = air density; \( C_{x} \) = shape factor; \( \alpha_{0} \) = drag force coefficient; and \( B \) = building width.

**Example I: Tall Building**

An example building with the dimensions \( H = 200 \) m, \( B = 50 \) m, and \( D = 40 \) m, is used to show the distribution of load effects along the building height. The structural data are as follows: \( f_{1} = 0.2 \) Hz; \( \xi_{1} = 0.01; \beta = 1.5; m_{0} = 500,000 \) kg/m; and \( C_{g1} = 1.3 \). The wind data are as follows: \( U_{10} = 30 \) m/s; \( \alpha = 0.15 \); \( \alpha_{f} / U_{10} = 0.2 \); and a Davenport type spectrum is used.

The results are illustrated in Fig. 1, including the load effects by the GLF method. Note that the resonant equivalent static wind load varies as a power of \( 2\beta \) in the GLF method, but in the proposed formulation, it varies as a power of \( \beta \). Although the first mode displacement response is identical in the two formulations, the same cannot be said for other load effects. For this example, as far as the base shear force is concerned, the background and resonant responses are 98.1% and 134.3% of the theoretical estimates. It is also noteworthy that the latter shows that the GLF method is on the conservative side. However, for the resonant force at the top floor, this value is 71.0%, which is 29.0% on the unsafe side.

**EQUIVALENT STATIC WIND LOADS ON HORIZONTAL STRUCTURES**

Equivalent static wind loads on horizontal structures can be derived using the above procedure given for vertical structures. For these structures, the expressions are generally much simpler, because it usually can be assumed that the wind speed does not vary along the span. Using the buffeting theory of Scanlan and Gade (1977), expressions for equivalent buffeting wind loads in the vertical mode on horizontal decks are summarized in the following. This method can easily be extended to other horizontal structures and also used for qualifying torsional load effects.

The mean wind loading is given by

\[ P_{x}(s) = \frac{1}{2} \rho C_{x} \alpha_{0} B \]  

in which \( P_{x}(s) = \) mean wind load in the vertical direction at
span x; h = vertical coordinate; \( \bar{U} \) = mean wind velocity at the bridge height (horizontal); \( C_s \) = lift coefficient of the deck section; \( \alpha_0 \) = angle of attack; and \( B \) = width of the bridge deck.

The background equivalent wind load can be computed by

\[
P_{ba}(x) = g_b Q(x) \sigma_r(x)
\]

where

\[
Q(x) = \frac{1}{\sqrt{2\pi}} \int_0^\infty \int_0^L \left( \frac{x}{p(x)p(x)} \right)^{1/2} \cdot \sigma_r(x)
\]

\[
\sigma_r(x) = \frac{\rho \bar{U}^2}{2m} \left( \int_0^\infty S_L(f) df \right)^{1/2}
\]

in which \( P_{ba}(x) \) = peak background equivalent wind load pertaining to a certain response governed by the influence coefficient function \( h(x) \); \( g_b \) = background peak factor; \( Q(x) \) = load-response-correlation coefficient; \( \sigma_r(x) \) = RMS fluctuating wind force at span x; \( L \) = span of the bridge; \( p(x)p(x) \) = covariance of the fluctuating wind force between position \( x_i \) and \( x_j \); \( S_L(f) \) = spectrum of fluctuating applied wind force; \( C_L \) = slope of the lift coefficient curve versus the angle of attack; \( A \) = deck projected area normal to the wind per unit span; \( C_D \) = drag coefficient; and \( S_L(f) \) and \( S_S(f) \) = spectra of the horizontal and vertical fluctuating wind velocity, respectively.

The resonant equivalent wind loads are represented by the inertial loads

\[
P_{rb}(x) = g_r \frac{\rho \bar{U}^2}{\phi_r} \varphi(x) \sqrt{\frac{\pi f_r^2}{4 \phi_r^2}} |J(f)|^2 S_L(f)
\]

\[
\bar{\zeta}_m = \zeta_m - \frac{\rho \bar{U}^2}{2m} \bar{H}(K)
\]

\[
\left| J(f) \right|^2 = \int_0^L \int_0^L \phi(x_1)\phi(x_2) \exp \left( -\frac{cf}{U} |x_1 - x_2| \right) dx_1 dx_2
\]

in which \( P_{rb}(x) \) = resonant equivalent wind load in the \( r \)th mode; \( \phi_r \) = natural frequency in the \( r \)th mode; \( |J(f)|^2 \) = joint acceptance; \( \zeta_m \) = resultant damping ratio; \( \zeta_m \) = mechanical damping ratio in the \( r \)th mode; \( m \) = mass per unit span; \( H^*(K) \) = aerodynamic derivative of the deck section; and \( K = BF/U \) is the reduced frequency.

Because of non-uniform distributions of the load components along the span, it is suggested that the resultant peak load effects, rather than the peak loading, be combined by the SRSS rule as

\[
\bar{R}(z) = \bar{R}(z) + \sqrt{(\bar{R}_{ba}(z))^2 + \sum (\bar{R}_{rb}(z))^2}
\]

where \( \bar{R}_{ba} \) and \( \bar{R}_{rb} \) = mean, background, and resonant response obtained with the static structural analysis by employing the above load components separately.

**Example II: Suspension Bridge**

Deck data from the Humen Bridge, which is located in Guangdong, China, is used for this illustration: \( L = 880 \) m (main span); \( B = 35.6 \) m; \( A = 3.012 \) m; \( n_s = 23,240 \) kg/m; \( f_i = 0.1117 \) Hz, asymmetrical; \( f_i = 0.1715 \) Hz, symmetrical; and \( \zeta_m = 0.5% \) for all modes. The aerodynamic data is as follows: \( C_L(0) = 0; C_L'(0) = 4.57; C_D = 0.812; H^*(K) = -4.0 \), where \( K_i = Bf_i/U \); and \( H^*(K) = -2.7 \), where \( K_i = Bf_i/U \). The wind data used for this example are: \( Z_0 = 0.01 \) m; \( \alpha = 0.12 \); and \( \bar{U} = 50 \) m/s.

Because the mean wind force is zero and the mode shape is asymmetrical, the equivalent static wind loading for the Humen Bridge cannot be evaluated using the GLF method. The background and resonant equivalent wind loads in the first and second modes and associated displacement responses are computed using the given procedure. The results are illustrated in Fig. 2. It is noteworthy that the contribution of the background response is not insignificant even for this relatively flexible bridge.

**CONCLUDING REMARKS**

Since the traditional GLF method has intrinsic shortcomings in defining equivalent static buffeting wind loads on structures, this note presents a theoretical formulation that provides a succinct description of the equivalent static buffeting wind loads. The proposed method can be extended to problems other than the buffeting, e.g., the across-wind and torsional load effects. In these cases, information about the correlation of the aerodynamic pressure distribution is needed.

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APPENDIX. REFERENCES


