EQUIVALENT STATIC BUFFETING LOADS ON STRUCTURES

By Yin Zhou,[1] Ahsan Kareem,[2] Member, ASCE, and Ming Gu,[3] Member, ASCE

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:

$$\hat{P}(z) = G\bar{P}(z) \tag{1}$$

where the constant G is formulated based on the displacement

$$G = \hat{Y}(z)/\bar{Y}(z) \tag{2}$$

in which $\hat{Y}(z)$, $\hat{P}(z)$, $\bar{Y}(z)$, and $\bar{P}(z)$ = peak and mean displacement responses 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. Another 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 cantilever 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 $m(z) = m_0$; first mode shape $\varphi_1(z) = (z/H)^{\beta}$; β is constant; and the natural frequency and damping ratio in the first mode are f_1 and ζ_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:

$$\bar{P}(z) = \frac{1}{2}\rho\bar{U}_H^2 C_d B(z/H)^{2\alpha} \tag{3}$$

where $\bar{U}_H$ = mean wind velocity at the top height of the building; ρ = air density; α = 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

$$\hat{P}_B(z) = g_B Q(z)\sigma_p(z) \tag{4}$$

$$Q(z) = \frac{\int_0^{\infty}\int_0^{H}\int_0^{B}\int_0^{B}\overline{p(z,x_1)p(z_1,x_2)}\,i(z_1)\,dx_1\,dx_2\,dz_1\,df}{\left(\int_0^{\infty}\int_0^{H}\int_0^{H}\int_0^{B}\int_0^{B}\overline{p(z_1,x_1)p(z_2,x_2)}\,i(z_1)i(z_2)\,dx_1\,dx_2\,dz_1\,dz_2\,df\right)^{1/2}\cdot\sigma_p(z)} \tag{5}$$

$$\sigma_p(z) = \rho\bar{U}_H C_d B\left(\int_0^{\infty} S_u(f)\,|J_H(f)|^2\,df\right)^{1/2}(z/H)^{\alpha} \tag{6}$$

where g_B = background peak factor (Zhou and Kareem, manu-

[1]Res. Assoc., NatHaz Modeling Lab., Dept. of Civ. Engrg. and Geological Sci., Univ. of Notre Dame, Notre Dame, IN 46556.

[2]Prof., NatHaz Modeling Lab., Dept. of Civ. Engrg. and Geological Sci., Univ. of Notre Dame, Notre Dame, IN.

[3]Prof., Dept. of Bridge Engrg., Tongji Univ., Shanghai, 200092, P.R. China.

Note. Associate Editor: Timothy A. Reinhold. Discussion open until January 1, 2001. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this technical note was submitted for review and possible publication on October 29, 1997. This technical note is part of the ***Journal of Structural Engineering***, Vol. 126, No. 8, August, 2000. ©ASCE, ISSN 0733-9445/99/0008-0989-0992/$8.00 + $.50 per page. Technical Note No. 16852.

script in preparation, 2000); $S_u(f)$ = spectrum of the fluctuating wind velocity; $i(z) = i_c(z/H)^{\beta_0}$ = influence function in which i_c and β_0 are constants; $|J_H(f)|^2 = (1/B^2)\int_0^B\int_0^B R_x(x_1, x_2, f)dx_1 dx_2$ is the horizontal joint acceptance function; and $R_x(x_1, x_2, f) = \exp(-c|x_1 - x_2|)$, where $c = 8\ fh/\bar{U}(h), h = 0.6H$, is the horizontal coherence function of the surface fluctuating wind pressure. Eqs. (4)–(6) show that the distribution of the background equivalent wind load is independent of the mode shape, β, but depends on the load effect being evaluated, β_0.

For a wind excited structure, only the contribution of the resonant response in the first mode is typically considered. Using random vibration theory, the resonant equivalent wind load can be expressed in terms of the inertial force as follows:

$$\hat{P}_{R1}(z) = m(z)(2\pi f_1)^2\hat{D}_{R1}(z)$$

$$= \frac{g_R\rho\bar{U}_H C_d B(2\beta + 1)}{(1 + \alpha + \beta)}|J_H(f_1)||J_z(\alpha, \beta, f_1)|\sqrt{\frac{\pi f_1}{4\zeta_1}S_u(f_1)}\left(\frac{z}{H}\right)^{\beta} \quad (7)$$

where $\hat{D}_{R1}$ = first mode displacement; $g_R = \sqrt{2\ln(f_1T)} + 0.5772/\sqrt{2\ln(f_1T)}$ is the resonant peak factor; and $|J_z(\alpha, \beta_0, f)|^2 = [(1 + \alpha + \beta_0)^2/H^2]\int_0^H\int_0^H (z_1/H)^{\alpha+\beta_0}(z_2/H)^{\alpha+\beta_0}R_z(z_1, z_2, f)dz_1dz_2$ is the vertical joint acceptance function, where $R_z(z_1, z_2, f) = \exp(-c|z_1 - z_2|)$. It is noted that the distribution of the resonant equivalent wind load follows a power of β. Details of the derivations of (4) and (7) can be found in Zhou and Kareem (1999).

The associated structural responses are also summarized here. Since the load components have different distributions along the building height as outlined above, it is suggested that only the resultant peak response should be combined by the SRSS rule as given below (Zhou and Kareem, manuscript in preparation, 2000):

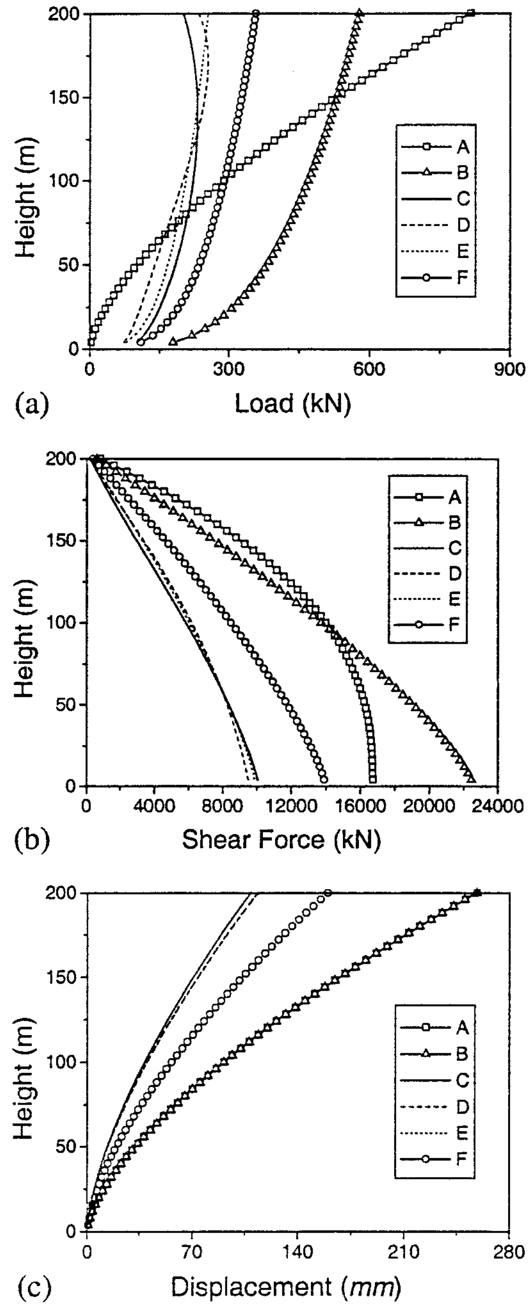
$$\hat{r}(z) = \bar{r}(z) + \sqrt{(\hat{r}_B(z))^2 + (\hat{r}_{R1}(z))^2} \quad (8)$$

where $\bar{r}$, $\hat{r}_B$, and $\hat{r}_{R1}$ = mean, background, and resonant response components obtained from the static structural analysis by employing the above load components separately.

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: $f_1 = 0.2\ Hz$; $\zeta_1 = 0.01$; $\beta = 1.5$; m_0 = 500,000 kg/m; and $C_d = 1.3$. The wind data are as follows: $\bar{U}_{10}$ = 30 m/s; α = 0.15; $\sigma_u/\bar{U}_{10} = 0.2$; 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 2α in the GLF method, but in the proposed formulation, it varies as a power of β. 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.



A = Actual Resonant Equivalent Wind Load
B = Resonant Equivalent Wind Load by GLF Method
C = Background Equivalent Wind Load for Base Shear Force
D = Background Equivalent Wind Load for First Mode Displacement Response
E = Background Equivalent Wind Load by GLF Method
F = Mean Wind Force

FIG. 1. Wind Loading and Effects on Tall Building

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

$$\bar{P}_h(x) = 1/2\rho\bar{U}C_L(\alpha_0)B \quad (11)$$

in which $\bar{P}_h(x)$ = mean wind load in the vertical direction at span x; h = vertical coordinate; $\bar{U}$ = mean wind velocity at the bridge height (horizontal); C_L = lift coefficient of the deck section; α_0 = angle of attack; and B = width of the bridge deck.

The background equivalent wind load can be computed by

$$\hat{P}_{hB}(x) = g_B Q(x)\sigma_p(x) \tag{12}$$

$$Q(x) = \frac{\int_0^{\infty}\int_0^{L} \overline{p(x_1)p(x)}\,i(x_1)dx_1\,df}{\left(\int_0^{\infty}\int_0^{L}\int_0^{L} \overline{p(x_1)p(x_2)}\,i(x_1)i(x_2)dx_1\,dx_2\,df\right)^{1/2} \cdot \sigma_p(x)} \tag{13}$$

$$\sigma_p(x) = \rho\bar{U}B\left(\int_0^{\infty} S_L(f)df\right)^{1/2} \tag{14}$$

$$S_L(f) = C_L^2 S_u(f) + \frac{1}{4}\left(C_L' + \frac{A}{B}C_D\right)^2 S_w(f) \tag{15}$$

in which $\hat{P}_{hB}(x)$ = peak background equivalent wind load pertaining to a certain response governed by the influence coefficient function $i(x)$; g_B = background peak factor; $Q(x)$ = load-response-correlation coefficient; $\sigma_p(x)$ = RMS fluctuating wind force at span x; L = span of the bridge; $\overline{p(x_1)p(x_2)}$ = covariance of the fluctuating wind force between position x_1 and x_2; $S_L(f)$ = 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 $S_u(f)$ and $S_w(f)$ = spectra of the horizontal and vertical fluctuating wind velocity, respectively.

The resonant equivalent wind loads are represented by the inertial loads

$$\hat{P}_{hRi}(x) = g_R\frac{\rho\bar{U}B}{\phi_i}\varphi_i(x)\sqrt{\frac{\pi f_i}{4\bar{\zeta}_{hi}}|J(f_i)|^2 S_L(f_i)} \tag{16}$$

$$\bar{\zeta}_{hi} = \zeta_{hi} - \frac{\rho B^2}{2m}H_1^*(K) \tag{17}$$

$$|J(f)|^2 = \int_0^{L}\int_0^{L}\varphi_i(x_1)\varphi_i(x_2)\exp\left(-\frac{cf}{\bar{U}}|x_1 - x_2|\right)dx_1\,dx_2 \tag{18}$$

in which $\hat{P}_{hRi}$ = resonant equivalent wind load in the ith mode; $\phi_i = \int_0^L \varphi_i^2(x)dx$; $\varphi_i(x)$ = ith mode shape; f_i = natural frequency in the ith mode; $|J(f)|^2$ = joint acceptance; $\bar{\zeta}_{hi}$ = resultant damping ratio; ζ_{hi} = mechanical damping ratio in the ith mode; m = mass per unit span; $H_1^*(K)$ = aerodynamic derivative of the deck section; and $K = Bf/\bar{U}$ 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

$$\hat{r}_h(z) = \bar{r}_h(z) + \sqrt{(\hat{r}_{hB}(z))^2 + \sum(\hat{r}_{hRi}(z))^2} \tag{19}$$

where $\bar{r}_h$, $\hat{r}_{hB}$, and $\hat{r}_{hRi}$ = 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; m_0 = 23,240 kg/m; f_1 = 0.1117 H_z, asymmetrical; f_2 = 0.1715 H_z, symmetrical; and ζ_{hi} = 0.5% for all modes. The aerodynamic data is as follows: $C_L(0) = 0$; $C_L'(0) = 4.57$; $C_D = 0.812$; $H_1^*(K_1) = -4.0$, where $K_1 = Bf_1/\bar{U}$; and $H_1^*(K_2) = -2.7$, where $K_2 = Bf_2/\bar{U}$. The wind data used for this example are: Z_0 = 0.01 m; α = 0.12; and $\bar{U}$ = 50 m/s.

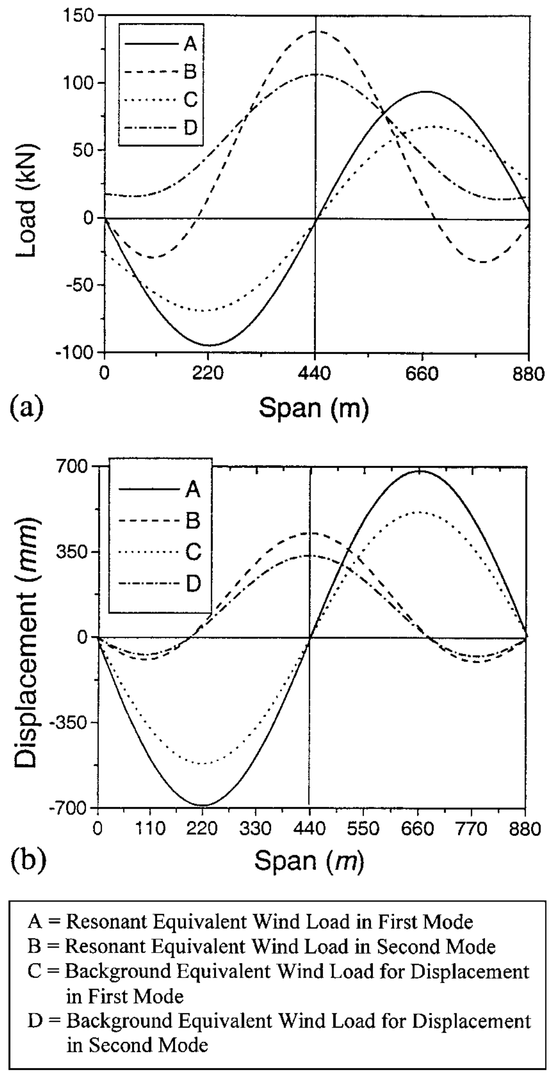


FIG. 2. Wind Loading and Effects on Humen Bridge

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.

ACKNOWLEDGMENT

The writers gratefully acknowledge the partial support from NSF grant numbers CMS 95-03779 and CMS 95-22145 for this study. This work was conducted under a cooperative research between the University of Notre Dame and Tongji University, Shanghai.

APPENDIX. REFERENCES

ASCE. (1996). ''Minimum design loads for buildings and other structures'' *ASCE 7-95*, New York.

Davenport, A. G. (1967). ''Gust loading factors.'' *J. Struct. Div.*, ASCE, 93(3), 11–34.

Kareem, A. (1982). ''Fluctuating wind loads on buildings.'' *J. Engrg. Mech. Div.*, ASCE, 108(6), 1086–1102.

Kasperski, M., and Niemann, H. J. (1992). ''The L.R.C. method—a general method of estimating unfavorable wind load distributions for linear and non-linear structures.'' *J. Wind Engrg. and Indust. Aerodyn.*, 41–44, 1753–1763.

Scanlan, R. H., and Gade, R. H. (1977). ''Motion of suspended bridge spans under gusty wind.'' *J. Struct. Div.*, ASCE, 103(9), 1867–1883.

Simiu, E. (1980). ''Revised procedure for estimating along-wind response.'' *J. Struct. Div.*, ASCE, 106(1), 1–10.

Solari, G. (1993a). ''Gust buffeting. I: Peak wind velocity and equivalent pressure.'' *J. Struct. Engrg.*, ASCE, 119(2), 365–382.

Solari, G. (1993b). ''Gust buffeting. II: Dynamic alongwind response.'' *J. Struct. Engrg.*, ASCE, 119(2), 383–398.

Solari, G., and Kareem, A. (1998). ''On the formulation of ASCE7-95 gust effect factor.'' *J. Wind Engrg. and Indust. Aerodyn.*, 77–78, 673–684.

Vickery, B. J. (1970). ''On the reliability of gust loading factors.'' *Proc., Tech. Mtg. Concerning Wind Loads on Build. and Struct.*, National Bureau of Standards, Washington, D.C., 296–312.

Zhou, Y., Gu, M., and Xiang, H. F. (1999a). ''Along-wind static equivalent wind loads and response of tall buildings. I: Unfavorable distributions of static equivalent wind loads.'' *J. Wind Engrg. and Indust. Aerodyn.*, 79(1–2), 135–150.

Zhou, Y., Kareem, A., and Gu, M. (1999b). ''Gust loading factors for design applications.'' *Proc., 10th ICWE*, Danish Maritime Institute, Lyngby, Denmark, 169–176.

Zhou, Y., and Kareem, A. (1999). ''Equivalent static buffeting loads on structures.'' *Tech. Rep. No. ND99-001*, Dept. of Civ. Engrg. and Geological Sci., University of Notre Dame, Notre Dame, Ind.