Prediction of buffeting response of long span bridges to transient nonstationary winds

Xinzhong Chen

Wind Science and Engineering Research Center, Department of Civil and Environmental Engineering, Texas Tech University, Lubbock, TX, USA

ABSTRACT: This paper presents a general frequency domain framework for predicting buffeting response of long span bridges to transient nonstationary winds. The wind speed field is described in terms of deterministic time-varying mean and evolutionary random fluctuations characterized by their evolutionary spectra. The static, self-excited and buffeting forces on bridges are modeled in static force coefficients, flutter derivatives and admittance functions with consideration of transient nonstationary winds. Formulations are presented for estimating the time-varying mean, evolutionary spectrum, variance and extreme response. The general characteristics of transient buffeting response are discussed using a long span suspension bridge as an example. The effectiveness and accuracy of the proposed framework is also confirmed through time domain Monte Carlo simulations.

KEYWORDS: Buffeting response; Nonstationary response; Bridge aerodynamics; Long span bridges; Wind loading.

1 INTRODUCTION

Current analysis frameworks for estimating wind-induced buffeting response of structures are generally based on the assumption of stationary wind excitations. The mean wind speed within a given time duration, say, 1 hour, is considered as a constant, and the wind fluctuation around its mean is modeled as a stationary random process. The stationarity assumption faces challenges in the cases such as hurricane or typhoon winds and thunderstorm downbursts, where the magnitude and direction of mean wind speed may change with time relatively rapidly, and the random wind turbulence may also show nonstationary characteristics.

Davenport and King (1993) illustrated the influence of storm duration on 'build-up' of single torsional modal response of Golden Gate Bridge, and pointed out that the maximum response in a given storm may be greater than it would be in other storm having a higher wind speed but short duration. Chen (2008) developed an analysis framework for quantifying alongwind tall building response under transient nonstationary winds, and investigated the influence of time-varying mean wind speed, mean wind speed profile, and spatial correlation of wind fluctuations on building response. Kwon and Kareem (2009) introduced a general gust-front factor accounting for the extreme response of buildings under transient winds. The theory concerning structural response to nonstationary excitations (e.g., Lin and Cai 1995; Lutes and Sarkani 2004) has been used in earthquake engineering (e.g., Conte and Peng 1997; Michaelov et al. 2001; Barbato and Vasta 2010), but its applications in wind engineering has been very limited.

This paper presents a general frequency domain framework for predicting buffeting response of long span bridges to transient nonstationary winds. Formulations are presented for modeling static, self-excited and buffeting forces and for estimating the time-varying mean, variance and extreme response. The general characteristics of transient buffeting response are discussed using a long span suspension bridge as an example. Time domain Monte Carlo simulations are also performed to confirm the effectiveness and accuracy of the proposed framework.
2 THEORETICAL FRAMEWORK

2.1 Equations of bridge motion

A long span bridge subjected to wind excitation with the mean wind direction normal to the bridge deck axis is considered. At the spanwise location $x$, the deterministic time-varying mean wind speed is denoted by $U(x,t)$ with its random fluctuation components $u'(x,t)$ and $w'(x,t)$ in horizontal and vertical directions, respectively. $u'(x,t)$ and $w'(x,t)$ are regarded as zero mean evolutionary random processes with the underlying zero mean stationary random processes $u(x,t)$ and $w(x,t)$ and complex-valued deterministic modulation functions $g_u(x,\omega,t)$ and $g_w(x,\omega,t)$, respectively. $u'(x,t)$ and $u(x,t)$ are expressed in the general form of a Fourier-Stieltjes integral as

$$u'(x,t) = \int_{-\infty}^{\infty} g_u(x,\omega,t)e^{i\omega t} d\Theta_u(x,\omega)$$

$$u(x,t) = \int_{-\infty}^{\infty} e^{i\omega t} d\Theta_u(x,\omega)$$

(1)

where $d\Theta_u(x,\omega)$ is complex-valued zero mean orthogonal increment random process

$$E[d\Theta_u(x,\omega)] = 0; \quad E[d\Theta_u^*(x,-\omega)] = E[d\Theta_u^*(x,\omega)]$$

(2)

$E[...]$ denotes the expectation or ensemble average; $\delta(...)$ represents the Dirac delta function; $S_u(x,\omega) = S_u(x,\omega,0)$ and $S_u(x,\omega)$ are the power spectral density (PSD) of $u(x,t)$ and cross PSD of $u(x_1,t)$ and $u(x_2,t)$; $\omega$ is frequency; superscript * denotes complex conjugate operator; and $i = \sqrt{-1}$.

The evolutionary PSD (EPSD) of $u'(x,t)$ and cross EPSD of $u'(x_1,t)$ and $u'(x_2,t)$ are then given as

$$S_u(x,\omega,0) = |g_u(x,\omega,0)|^2 S_u(x,\omega); \quad S_u'(x_1,\omega_1,\omega_2) = g_u'(x_1,\omega_1)g_u^*(x_2,\omega_2)S_u(x_1,\omega_2)$$

(3)

where $g_u'(x,-\omega,0) = g_u^*(x,\omega,0)$. Similar formulations for $w'(x,t)$ and $w(x,t)$ can be given in terms of $d\Theta_u(x,\omega)$, $g_w(x,\omega,0)$, $S_w(x,\omega,0)$ and $S_w(x_1,\omega_1,\omega_2)$.

The aerodynamic forces on bridge deck, i.e., lift (downward), drag (downwind) and pitching moment (nose-up), can be separated into static (mean), self-excited and buffeting force components. It is assumed that the variation rate of time-varying mean wind speed is low enough such that the effects of transient aerodynamics can be neglected. Accordingly, the aerodynamic forces can be given in the same formulations as traditional time-invariant mean wind speed case, but with consideration of time-varying mean wind speed and evolutionary wind fluctuations. For instance, the static, self-excited and buffeting components of pitching moment per unit length of bridge deck are given as (e.g., Scanlan 1993; Chen and Kareem 2002)

$$M_s(x,t) = \frac{1}{2} \rho U^2(x,t)B^2 C_M$$

(4)

$$M_{se}(x,t) = \frac{1}{2} \rho U^2(x,t)B^2 \left( kA_1^* \frac{\dot{h}}{U} + kA_2^* \frac{\dot{b}a}{U} + k^2 A_1^* \alpha + k^2 A_2^* \frac{\dot{h}}{b} + kA_2^* \frac{\dot{p}}{U} + k^2 A_1^* \frac{\dot{p}}{b} \right)$$

(5)

$$M_b(x,t) = \frac{1}{2} \rho U(x,t)B^2 \int_{-\infty}^{\infty} \left[ 2C_M \chi_{mb}(x,\omega,t)g_u(x,\omega,t)e^{i\omega t} d\Theta_u(x,\omega) \
+ 2C_M' \chi_{mb}(x,\omega,t)g_w(x,\omega,t)e^{i\omega t} d\Theta_w(x,\omega) \right]$$

(6)
where \( \rho \) is air density; \( B = 2b \) is bridge deck width; \( C_M \) is static pitching moment coefficients; \( C_M = dC_M / dx \); \( A_i \) \((i = 1 - 6)\) are flutter derivatives, which are function of reduced frequency \( k = ob / U(x,t) \); \( h(x,t) \), \( p(x,t) \) and \( \alpha(x,t) \) are bridge deck dynamic displacements around the statically deformed equilibrium at the spanwise location \( x \) in acrosswind, alongwind and torsional directions; \( \lambda_M \) and \( \lambda_W \) are aerodynamic admittance functions.

It is assumed that the time variation rates of mean wind speed and mean wind loading are low compared to the natural frequencies of the bridge, therefore the resulting bridge response can be calculated through quasi-static analysis. An iterative calculation procedure is need to solve this statically nonlinear problem as the static force coefficients are functions of angle of attack which is influenced by bridge deck rotation. The dynamic buffeting response around the statically deformed equilibrium is expressed in generalized modal responses. Following the finite element procedure, the equations of bridge motion can be expressed as follows in the generalized modal displacement vector \( \mathbf{q} \) (e.g., Chen et al. 2000a and b)

\[
M\ddot{\mathbf{q}} + C\dot{\mathbf{q}} + K\mathbf{q} = \mathbf{Q}_w + \mathbf{Q}_b
\]

\[
M = \text{diag}[m_j]; \quad C = \text{diag}[2m_j\xi_j\omega_j]; \quad K = \text{diag}[m_j\omega_j^2]
\]

\[
\mathbf{Q}_w = \mathbf{A}_w(\omega,t)\mathbf{q}(t) + \mathbf{A}_f(\omega,t)\dot{\mathbf{q}}(t)
\]

\[
\mathbf{Q}_b = \int_{-\infty}^{\infty} \left( \mathbf{A}_w(\omega,t)e^{j\omega t}d\Theta_w(\omega) + \mathbf{A}_f(\omega,t)e^{j\omega t}d\Theta_f(\omega) \right)
\]

where \( M, C, K \) are generalized mass, damping and stiffness matrices; \( m_j, \xi_j, \omega_j \) are j-th generalized mass, damping ratio and frequency; \( \mathbf{Q}_w \) and \( \mathbf{Q}_b \) are generalized self-excited and buffeting force vectors; \( \mathbf{A}_w(\omega,t) \) and \( \mathbf{A}_f(\omega,t) \) are aerodynamic stiffness and damping matrices; \( \mathbf{A}_w(\omega,t) \) and \( \mathbf{A}_f(\omega,t) \) are buffeting force matrices; \( d\Theta_w(\omega) \) and \( d\Theta_f(\omega) \) are orthogonal increment random processes associated with horizontal and vertical wind fluctuation vectors, characterized by the spectral matrices, i.e.,

\[
E[d\Theta_w(\omega)d\Theta_w^*(\omega)]=S_w(\omega)\delta(\omega-\omega_1)d\omega; \quad E[d\Theta_f(\omega)d\Theta_f^*(\omega)]=S_f(\omega)\delta(\omega-\omega_1)d\omega
\]

and \( T \) denotes the matrix transpose operator.

The equations of bridge motion can be represented in the state-space format:

\[
\dot{\mathbf{Y}} = \mathbf{A}(\omega,t)\mathbf{Y} + \mathbf{BQ}_b
\]

\[
\mathbf{Y} = \begin{bmatrix} \mathbf{q} \\ \mathbf{\dot{q}} \end{bmatrix}; \quad \mathbf{A}(\omega,t) = \begin{bmatrix} 0 & \mathbf{I} \\ -M^{-1}[K - \mathbf{A}_f(\omega,t)] & -M^{-1}[C - \mathbf{A}_f(\omega,t)] \end{bmatrix}; \quad \mathbf{B} = \begin{bmatrix} \mathbf{0} \\ M^{-1} \end{bmatrix}
\]

At a given time instant, "frozen time", i.e., a given wind speed, the complex modal properties can be quantified through the solution of following complex eigenvalue problem:

\[
\lambda_j\Gamma_j = \mathbf{A}(\omega_j,t)\Gamma_j; \quad \lambda_j = -\xi_j\omega_j + i\omega_j\sqrt{1-\xi_j^2}; \quad \Gamma_j = \begin{bmatrix} \Phi_j \\ \lambda_j\Phi_j \end{bmatrix}
\]

where \( \lambda_j \) and \( \Phi_j \) are j-th complex eigenvalue and eigenmode (mode shape) with their complex conjugates \( \lambda_{j,N} = \lambda_j \) and \( \Phi_{j,N} = \Phi_j \) (where \( N \) is the total number of modes considered); \( \omega_j \) and \( \xi_j \) are j-th complex modal frequency and damping ratio. An iterative calculation is needed for the solution of this eigenvalue problem as \( \mathbf{A}(\omega,t) \) involves the unknown frequency.
The corresponding eigenmode of \( A^T(\omega, t) \) is expressed as

\[
\lambda_j \Psi_j = A^T(\omega_j, t) \Psi_j; \quad \Psi_j = \begin{bmatrix} \Psi_{j,\omega} \\ \Psi_{j,d} \end{bmatrix}
\]  

(15)

It is noted that each of the complex eigenvalues and modal shapes are determined using different system matrix defined at the corresponding modal frequency, i.e., \( A(\omega_j, t) \). Therefore, theoretically the orthogonal condition between different modal shapes does not exist, but can be considered to be approximately valid without introducing a noticeable influence on the system behavior. Accordingly, the equations of bridge motion can be represented in an equivalent frequency-independent format with \( A_{\alpha}(t) \) replacing \( A(\omega, t) \) (Chen 2006). This equivalent linear time-variant (LTV) bridge aeroelastic system is uniquely defined by the complex modal properties. Denote

\[
\Gamma = [\Gamma_1 \ \Gamma_2 \ \cdots \ \Gamma_{2N}] = [\Phi \ \Phi_A]; \quad \Phi = [\Phi_1 \ \Phi_2 \ \cdots \ \Phi_{2N}]; \quad \Psi = [\Psi_1 \ \Psi_2 \ \cdots \ \Psi_{2N}] = [\Psi_{\omega} \ \Psi_d]
\]  

(16)

the orthogonal condition leads to

\[
\Psi^T = \Gamma^{-1}; \quad \Gamma^{-1} A_{\alpha} \Gamma = \Lambda = \text{diag}[\lambda_j]
\]  

(17)

which can be used to determine \( A_{\alpha}(t) \) if needed.

2.2 Frequency domain buffeting response analysis

Introducing a transformation \( Y(t) = P(t)R(t) \), the state-space equation of motion becomes

\[
\dot{R}(t) = P^{-1}(t)[A_{\alpha}(t)P(t) - \dot{P}(t)]R(t) + P^{-1}(t)BQ(t)
\]  

(18)

If \( P(t) \) is a fundamental matrix, i.e., it satisfies the dynamic eigenvalue equation \( \dot{P}(t) = A_{\alpha}(t)P(t) \), the state-space equation leads to

\[
Y(t) = \Xi(t, t_0)Y(t_0) + \int_{t_0}^{t} \Xi(t, \tau)BQ_{\gamma}(\tau)d\tau
\]  

(19)

where \( \Xi(t, \tau) = P(t)P^{-1}(\tau) \) is state transition matrix; and \( Y(t_0) \) is the state vector at \( t_0 \).

The dynamic eigenvalue equation of the LTV system can be solved, for instance, using an approach introduced by Kloet and Neerhoff (2000), which is based on triangularization of system matrix by Riccati transformation. In this study, it is assumed that the variation rate of complex mode shape \( \Gamma(t) \) is relatively small, thus \( \Gamma(t) \approx 0 \), and the fundamental matrix can be approximately given by the "frozen time" complex eigenvalues and vectors as

\[
P(t) = \Gamma(t)e^{\int_{0}^{t} \lambda(\tau)d\tau}
\]  

(20)

When the system is from "at rest" at \( t = 0 \), the generalized displacement and velocity vectors are given as follows based on Eq. (19):

\[
q(t) = \int_{0}^{t} h_q(t, \tau)Q_{\gamma}(\tau)d\tau; \quad \dot{q}(t) = \int_{0}^{t} h_q(t, \tau)Q_{\gamma}(\tau)d\tau
\]  

(21)
\[ h_q(t, \tau) = \Phi(t)e^{\int_0^t A(t)dt} \Psi_q^T(\tau)M^{-1} \text{,} \quad h_y(t, \tau) = \frac{\partial h_q(t, \tau)}{\partial t} \approx \Phi(t)A(t)e^{\int_0^t A(t)dt} \Psi_q^T(\tau)M^{-1} \]  

where \( h_q(t, \tau) \) and \( h_y(t, \tau) \) are impulse response matrices for displacement and velocity.

The variance matrix of generalized displacement vector is then determined as

\[
R_q(t) = E[q(t)q^T(t)] = \int_0^T \int_0^T h_q(t, t_1)R_{q_h}(t_1, t_2)h_q^T(t_1, t_2)dt_1dt_2
\]

\[
R_{q_h}(t_1, t_2) = \int_{-\infty}^\infty S_{q_h}(\omega, t_1, t_2)e^{-i\omega(t_1-t_2)}d\omega
\]

\[
S_{q_h}(\omega, t_1, t_2) = A_{ju}(\omega, t_1)S_q(\omega)A_{ju}^T(\omega, t_2) + A_{ju}(\omega, t_1)S_w(\omega)A_{ju}^T(\omega, t_2)
\]

where \( R_{q_h}(t_1, t_2) \) and \( S_{q_h}(\omega, t_1, t_2) \) are the covariance matrix and cross EPSD between \( Q_h(t_1) \) and \( Q_h(t_2) \). It is assumed that \( u- \) and \( w- \)components of wind fluctuations are uncorrelated, while their correlation and cross spectrum can be readily incorporated in the formulations.

The EPSD matrix of the generalized displacement vector is determined as

\[
S_q(\omega, t) = H_u(\omega, t)S_q(\omega)H_u^T(\omega, t) + H_w(\omega, t)S_w(\omega)H_w^T(\omega, t)
\]

\[
H_u(\omega, t) = \int_0^T h_q(t, \tau)A_{ju}(\omega, \tau)e^{-i\omega(t-\tau)}d\tau; \quad H_w(\omega, t) = \int_0^T h_q(t, \tau)A_{ju}(\omega, \tau)e^{-i\omega(t-\tau)}d\tau
\]

and the integration of \( S_q(\omega, t) \) over the frequency gives \( R_q(t) \).

The time-varying variance and ESPD of any response of interest can be subsequently calculated. For a given response \( y(t) = D^Tq(t) \), these are

\[
\sigma^2(t) = R_y(t) = D^T R_y(t)D; \quad S_y(\omega, t) = D^T S_q(\omega, t)D
\]

and similar calculations for any response related to velocity response \( \dot{q}(t) \) can be made using the impulse response function matrix \( h_y(t, \tau) \).

The extreme value of \( v(t) \) within a time duration of \( T \), i.e., \( v_{\text{max}} = \max\{\nu(t), 0 \leq t \leq T\} \), can be calculated through upcrossing rate analysis. Under the assumption of Poisson crossings, the cumulative distribution function (CDF) of the extreme response is calculated as

\[
F_{\text{max}}(v) = \exp \left( -\int_0^T \mu(\nu, t)dt \right); \quad \mu(\nu, t) = \frac{1}{2\pi} \frac{\sigma(t)}{\sigma_y(t)} \exp \left( -\frac{(v-\bar{v}(t))^2}{2\sigma_y^2(t)} \right)
\]

where \( \mu(\nu, t) \) is the crossing rate at level \( \nu \); \( \sigma_y(t) \) is root-mean-square (RMS) value of \( \nu(t) \); and \( \bar{v}(t) \) is the time-varying mean response. The mean extreme corresponds to a non-exceeding probability of 0.57 when the extreme distribution can be further modeled as a Gumbel-type distribution. The mean extreme subtracting the maximum of mean and then derived by the maximum of RMS response is referred to as peak factor for the transient response.

2.3 Application to simplified cases

Consider the case where the mean wind speed and wind fluctuations are expressed as

\[
U(x, t) = U_{\text{max}}U_0(x)d_n(t); \quad u'(x, t) = u(x, t)d_n(t); \quad w'(x, t) = w(x, t)d_n(t)
\]

where \( d_n(t), d_n(t) \) and \( d_n(t) \) are time-varying modulation functions for the mean wind speed and wind fluctuations; \( U_{\text{max}} \) is maximum mean wind speed over time and space; \( U_0(x) \) defines the spanwise variation of the mean wind speed.
The PSDs and coherence functions of $u(x,t)$ and $w(x,t)$ are defined as

$$S_u(x_1, x_2, \omega) = \text{coh}_u(x_1, x_2, \omega)S_u(\omega)I^2_u(x_1)I^2_u(x_2); \quad \text{coh}_u(x_1, x_2, \omega) = \exp\left(-\frac{\lambda_u \omega |x_1 - x_2|}{2\pi U_R}\right)$$

$$S_w(x_1, x_2, \omega) = \text{coh}_w(x_1, x_2, \omega)S_w(\omega)I^2_w(x_1)I^2_w(x_2); \quad \text{coh}_w(x_1, x_2, \omega) = \exp\left(-\frac{\lambda_w \omega |x_1 - x_2|}{2\pi U_R}\right)$$

where $\lambda_u$ and $\lambda_w$ are decay factors; $I^2_u(x)$ and $I^2_w(x)$ are functions to reflect the spatial variations of PSDs of wind fluctuations; and $U_R = \max[U_0(x_1) + U_0(x_2)]/2$. It is also assumed that the admittance functions are functions of reduced frequency $k = \omega b / (U_{\max}U_0(x))$, thus are functions of spanwise location $x$ and frequency $\omega$, but are independent of time $t$.

Accordingly, the buffeting force matrices are calculated as

$$A_{bu}(\omega, t) = A_{nu0}(\omega)\bar{m}(t)d_u(t); \quad A_{bu}(\omega, t) = A_{nu0}(\omega)\bar{m}(t)d_u(t)$$

and the ESPD matrix of generalized displacement vector is estimated as

$$S_q(\omega, t) = H_{qbu}(\omega, t)S_{qbu}^{(0)}(\omega)H_{qbu}^T(\omega, t)$$

$$H_{qbu}(\omega, t) = \int_0^t h_q(t, \tau)d_m(\tau)e^{-j\omega(t-\tau)}d\tau; \quad H_{qbu}(\omega, t) = \int_0^t h_q(t, \tau)d_m(\tau)e^{-j\omega(t-\tau)}d\tau$$

It is noted that the transient characteristics of response is reflected by the time dependence of $H_{qbu}(\omega, t)$ and $S_q(\omega, t)$ in the case of the time-invariant mean wind speed and stationary wind fluctuations, i.e., $d_m(t) = \bar{m}(t) = \bar{d}_m(t) = 1$, $H_{qbu}(\omega, t)$ and $H_{qbu}(\omega, t)$, thus $S_q(\omega, t)$ and $R_q(t)$ approach to the steady-state values when time $t$ is sufficiently large. The framework presented in this study reduces to that for multimode coupled buffeting analysis of bridge aeroelastic system under stationary winds (e.g., Chen and Kareem 2000a; Chen 2006). It is also worthy of noting that when the aerodynamic inter-modal coupling is negligible, $H_{qbu}(\omega, t)$ and $H_{qbu}(\omega, t)$ are diagonal matrices. The transient buffeting response of each mode can be estimated separately and then combined by using the square-root-of-sum-of-squares (SRSS) combination rule.

3 NUMERICAL EXAMPLES AND DISCUSSIONS

3.1 A long span suspension bridge under nonstationary wind excitations

A suspension bridge with a main span of nearly 2000 m was used as an example. The first 15 natural modes with frequencies ranging from 0.003 to 0.2 Hz were considered for describing the dynamic behavior of the bridge. The modal damping ratio was assumed to be 0.0032. For simplicity and without loss of generality, only the aerodynamic forces acting on the bridge deck were considered. The self-excited lift and pitching moment were calculated based on the flutter derivatives derived from Theodorsen function. The self-excited drag was evaluated based on the quasi-steady theory. The admittance functions used were based on Davenport’s formula for drag with a decay factor of 8, and Sears function for lift and pitching moment. Two different joint acceptance functions were used for the buffeting force components associated with $u$- and $w$-components, respectively. The mean wind speed is uniform over the bridge span but with a time-varying amplitude, i.e.,
\[ U_0(x) = 1; \quad d_w(t) = \exp\left\{-\left(\frac{t-t_0}{2D_t}\right)^2\right\} \]  

(37)

where \( t_0 \) is the time instance at which the wind speed reach its maximum; and \( D_t \) is wind storm duration parameter defining the time variation of wind speed.

The modulation functions of \( u \)- and \( w \)-components of wind fluctuations were identical to that of the mean wind speed, i.e., \( d_u(t) = d_w(t) = d_w(t) \). The von Karman spectra were used to describe the underlying stationary wind fluctuations. The turbulence intensities and integral length scales were assumed to be 10 and 7.5\%, and 80 and 40 m, for \( u \)- and \( w \)-components, respectively. The spectral characteristics of wind fluctuations were uniform over the bridge, i.e., \( I_{u,0}(x) = I_{w,0}(x) = 1 \). The decay factors of the coherence functions for \( u \)- and \( w \)-components were \( \lambda_u = \lambda_w = 8 \). Fig.1 shows the time modulation function \( d_w(t) \) with \( t_0 = 300 \) sec, and \( D_t = 120, 180, 240 \) and 300 sec. A smaller value of \( D_t \) indicates a short duration of strong storm, i.e., a larger variation of mean wind speed.

### 3.2 Results and discussions

The modal frequencies, damping ratios and complex modal shapes at different mean wind speeds are calculated, which are used to calculate the impulse response function matrix. Fig.2 shows the modal damping ratios at different mean wind speeds. The coupled flutter onset velocity is estimated to be 69.4 m/s. Fig.3 displays the EPSD of the torsional displacement at main span center with \( U_{\text{max}} = 60 \) m/s, \( t_0 = 300 \) sec, and \( D_t = 120 \) sec. Fig.4 portrays the time-varying mean and RMS values of bridge deck displacements at center of main span. It is noted that the maximum RMS is delayed with respect to the time at which the mean wind speed reaches its maximum, which is due to transient effect of bridge dynamic system. The delay is more noticeable for lateral displacement that is dominated by first lateral mode with lower frequency and damping. Fig.5 shows the maximum values of static and RMS, and mean extreme (including static) response within 10 min at different spanwise location in terms of bridge deck vertical, lateral and torsional displacements. The results from ensemble average of 50 time domain simulation samples are also presented. The predictions in frequency and time domains are very agreeable.

The procedure for simulating the time history of transient response is essentially the same as that introduced in Chen et al. (2000b). The underlying stationary wind turbulence field is firstly generated based on their spectral characteristics using spectral representation method. The nonstationary wind turbulence field is then calculated by multiplying the modulation functions. The flutter derivatives, admittance functions and joint acceptance functions are represented as rational functions of reduced frequency. Based on the quasi-static assumption concerning the influence of time-varying mean wind speed on aerodynamics, the time histories of self-excited and
buffeting forces are generated and the resulting buffeting response is calculated using step-by-step solution of the dynamic equations.

Figure 4. Time-varying mean and RMS values of bridge deck displacements at main span center

Figure 5. Maximum values of static, RMS and mean extreme (including static) response under transient wind

Figure 6. Static, RMS and mean extreme (including static) response under constant mean wind speed

Figure 7. Influence of storm duration parameter on torsional displacement at main span center

Fig.6 shows the steady state buffeting response when mean wind speed is a constant of 60 m/s, which corresponds to traditional buffeting analysis. Fig.7 is the predicted maximum RMS responses and peak factors at different values of $D_t$ with $U_{max} = 40, 60$ and 70 m/s and $t_f = 300$ sec. It is obvious that the transient excitation results in reduced RMS response as compared to
steady state response. It is due to the lack of sufficient "build-up" time of response to reach the steady state level. At 70 m/s, the steady state buffeting response will be unstable as it exceeds the flutter onset speed. However, under the transient wind excitation the bridge shows stable response as the lack of sufficient "build-up" time of unstable response as shown in Fig.8. Take a single degree-of-freedom system as an example. The "build-up" time is the time required to make \[ \exp(-2\xi\omega t) \] close to zero in the case of positive damping, where \( \xi \) and \( \omega \) are the system damping ratio and frequency. Clearly, a large value of \( \xi \omega \), a shorter "build-up" time needed, and the RMS response becomes closer to the steady state value. Since the lateral displacement is dominated by lower frequency and with a lower modal damping, the transient effect is most significant. The build-up rate of unstable dynamic response is also proportional to the value of \( \xi \omega \). At higher wind speeds with larger negative damping, the buffeting responses will develop into significant amplitudes even under transient wind excitations.

The extreme value and peak factor of transient buffeting response are significantly lower than those of stationary response. For instance of case with \( U_{\text{max}} = 60 \text{ m/s}, t_c = 300 \text{ sec}, \) and \( D_t = 120 \text{ sec}, \) the peak factors of the bridge deck displacements in vertical, lateral and torsional directions at the main span center are 2.11, 1.43, and 2.08, respectively. These are 2.98, 2.68 and 3.07, respectively, in the case of steady state response under constant mean wind speed. The lower peak factor or extreme response is due to the fact that large response is only observed in a relatively short of time as compared to the stationary case where it occurs over the entire time duration. The crossing rate at a given response level is much lower than that of the state sate response.

4 CONCLUSIONS

This study presented a general frequency domain approach for quantifying buffeting response of long span bridges when subjected to transient nonstationary winds. The wind fluctuations were modeled as evolutionary random processes with time-varying mean. Formulations for calculating static, self-excited and buffeting forces were presented to account for the transient wind excitation. The bridge response to time-varying mean wind loading was determined through quasi-static analysis with an iterative procedure for solution of statically nonlinear problem. The dynamic bridge aeroelastic system was regarded as a linear time-variant system with slowly varying modal characteristics. The impulse response function matrix of the system was defined by the modal properties at "frozen time", which were determined through complex eigenvalue analysis at given mean wind speeds. Formulations were given for calculating the EPSD, time-varying RMS value, probability distribution of extreme response and peak factor. The traditional multi-mode coupled buffeting analysis framework to stationary wind excitation is the special case of the general framework presented in this study.

The analysis of buffeting response of the long span suspension bridge showed that transient wind excitation results in reduced RMS response attributed to the lack of sufficient "build-up" time of response to reach the steady state level. The lateral bridge deck displacement was more noticeably affected by the transient effect as it was dominated by the modal response with lower frequency and damping ratio. The duration of wind storm played a significant role in the generation of extreme buffeting response especially for very flexible structures with low frequency and damping. The extreme value and peak factor of transient response were much lower than those of steady-state response.

It is worthy of mentioning that the effects of transient aerodynamics, i.e., the potential changes in the aerodynamic force characteristics under transient winds, were neglected, which may be considerable in the case of rapidly varying transient winds. The information of transient
aerodynamics, when available, can be readily synthesized with the proposed approach and lead to a better prediction of structural response.

5 ACKNOWLEDGEMENTS

The support for this work was provided in part by NSF Grants CMMI-0824748 and CMMI-1029922, and is gratefully acknowledged.

6 REFERENCES