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A FRAMEWORK ON THE DYNAMIC RESPONSE OF TALL STRUCTURES TO NON-STATIONARY WIND USING DESIGN SPECTRUM

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Abstract

The use of design spectra is common practice in earthquake engineering design as opposed to wind-resisting design. Nonetheless, previous research has identified ways to map such technique to study wind effects on structures. The present investigation introduces a novel framework to deal with the dynamic performance of infrastructure subject to thunderstorm downbursts. It builds on recent developments that demonstrate that spectral analysis is applicable to wind engineering given various equivalences developed to quantify random loading generated through non-synoptic wind environment. The new framework is tested on a benchmark building via a simplified model, which demonstrates its validity. In this framework, the dynamic response of the multi-degree-of-freedom (MDOF) system subjected to non-stationary wind can be obtained assuming that the first mode of vibration is dominant when the mechanical properties of the structure and aerodynamic characteristics of the thunderstorm downburst are identified.

Keywords: Wind design spectrum, Wind-excited response, Thunderstorm downburst, Non-stationary wind, Multi-degree-of-freedom system

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1 INTRODUCTION

The potentially destructive effect of thunderstorm downbursts on structures raises concerns amongst societies for the safety and serviceability of infrastructure. The recorded highest microburst wind reached up to 150 mph, which is in the range of an EF3 tornado [1] with potentially severe damage to the buildings. The peak velocity of such events locates between 30m and 100m above the ground [2, 3], hence covering the height of critical civil engineering infrastructure. The study of the wind actions and damages on structures induced by the downburst outflows becomes crucial for the current wind engineering design.

Various researchers in the past have discussed the downburst wind load and the dynamic response of structures, see for example [4-9]. However, they acknowledge that the transient properties and limited measurements of this complex phenomena make its study uncertain and challenging, starting from the characterisation of the unsteady wind load. Considering the limited experimental measurements of the thunderstorm downburst, the wind field of these typical characteristics has been simulated through hybrid models [5, 6, 10], based on the theoretical velocity profiles. The state of the art is therefore limited to specific yet disperse techniques to estimate wind loading of thunderstorm downbursts.

The short duration and instantaneous properties of the downburst outflow lead to similarities with earthquake loads which are generally evaluated by the response spectra technique [11-13] – widely used in the past to deal with random vibrations, where the structural performance was combined with the inter-storey displacement and roof displacement ductility of the buildings. The proposed framework merges the response spectral technique applied in earthquake engineering with a refined characterisation of wind loads [14, 15]. It revisits the originally developed wind design spectra [16, 17] for the structural vibrations of regular buildings subjected to stationary wind to enhance that basic methodology to study non-stationary wind events.

2 MATHEMATICAL FRAMEWORK UNDERPINNING WIND DESIGN SPECTRA

Martinez-Vazquez [17] dealt with the wind design spectra for regular prismatic buildings subjected to stationary wind. The method requires the generalisation of force exerted by wind, structural mass, and stiffness, as well as the characterisation of partially correlated wind fields and its interaction with bluff bodies, to then apply classic spectral techniques for generating design spectra. The design spectra could be used to analyse MDOF systems via modal analysis, which unfolds the response of the oscillators (implicit in the design spectra) to capture the contribution of higher-order modes. The following paragraphs describe how the original formulation by Martinez-Vazquez [17] was tailored to thunderstorm downbursts.

The method departs from establishing the physical relationship between force and acceleration as established by Newton's Second Law on a vertical slender structure – see Eq. (1)-(2). The relationship enables the spectra of the input acceleration induced by the wind on a MDOF system, to be determined. The force spectrum being the scaled wind power spectrum.

$$S_F(z,n) = q^2(z) S_{\tilde{u}'}(z,n) \tag{1}$$

$$S_A(z,n) = \left(\frac{q(z)}{m}\right)^2 S_{\tilde{u}'}(z,n) \tag{2}$$

In these equations, S_F is the force spectra; $S_{\tilde{u}'}$ is the power spectra for reduced horizontal fluctuating velocity component $\tilde{u}'$, i.e., $nS_{\tilde{u}'}(z,n) = \frac{18f}{[1+27f]^{5/3}}$, where $f = nz/\overline{U}_{max}(z)$ [18] and $\overline{U}_{max}$ is the maximum value of the slowly varying mean velocity; S_A is the input acceleration spectra, i.e., the acceleration imparted by the wind to the structural system; m is the mass of the structure excited by the wind force; n is the frequency of gust wind; z is the height above the ground; q is a force factor, expressed as,

$$q(z) = c_D \rho A \bar{U}_{max}^2(z)[1 + 0.5\bar{I}_u(z)]\bar{I}_u(z) \tag{3}$$

In Eq. (3), c_D is the drag coefficients, ρ is the air density, A is the air exposed to wind, $\bar{I}_u$ is the slowly varying mean turbulence intensity. It needs to note that the force factor is purely induced by fluctuating turbulence where the fluctuating force $f' = 0.5c_D\rho A(2\bar{U}u' + u'^2)$. In this case, The second-order term of the fluctuating turbulence, u'^2, is replaced by $\sigma_u u'$ to simplify the calculation process, i.e., $2\bar{U}u' + u'^2 = (2\bar{U} + u')u' \approx (2\bar{U} + \sigma_u)\sigma_u\tilde{u}' \approx (2\bar{U} + I_u\bar{U})I_u\bar{U}\tilde{u}'$.

Considering the spatial correlation of wind gusts on MDOF system, the normalized cross spectra proposed by Davenport [19] is adopted, given in Eq. (4).

$$\chi(z,n) = exp\left\{-\frac{n}{1/2[\bar{U}_{max}(z_i)+\bar{U}_{max}(z_j)]}\sqrt{(C_y\Delta_y)^2 + (C_z\Delta_z)^2}\right\} \tag{4}$$

In Eq. (4), Δ_y and Δ_z are the horizontal and vertical distances between two points, i, j, located at coordinates $\{y_i, z_i\}$ and $\{y_j, z_j\}$ respectively. C_y and C_z are non-dimensional decay constants along with the horizontal and vertical directions, in this case, assumed equal to 10. $\bar{U}_{max}(z_i)$ and $\bar{U}_{max}(z_j)$ represent the maximum value of the slowly varying mean velocity at height z_i and z_j, respectively. In this case, we adopted the suggested shape function for the vertical velocity profile of horizontal wind suggested by Wood, et al. [20].

It follows that, by combining the spectra of input acceleration given in Eq. (2) with the cross spectra of the horizontal turbulence component given in Eq. (4), the cross-spectrum of the input acceleration can be obtained, see Eq. (5).

$$S_{A_{ij}}(z_i, z_j, n) = \frac{1}{m^2}q(z_i)q(z_j)\sqrt{S_{\tilde{u}'}(z_i,n)S_{\tilde{u}'}(z_j,n)}\frac{1}{A^2}\chi(z_i, z_j, n) \tag{5}$$

The integration of Eq. (5) across the area exposed to wind flow, contributes to the power spectral density of the generalised input acceleration $S_{cu}(z_i, z_j, n)$, given in Eq. (6).

$$S_{cu}(z_i, z_j, n) = \iint_A \phi(z_i)\phi(z_j)S_{A_{ij}}(z,n)dy_i dy_j dz_i dz_j \tag{6}$$

In Eq. (6), $\phi(z)$ represents the fundamental modal shape at height z above the ground. This provides the acceleration inputted to a system, therefore the variance of the overall spectral response can be obtained by passing the signal through the transfer function, as shown in Eq. (7). The integration of this equation can be done in two parts, to separate the background and resonant response components. This is expressed in Eq. (8) and Eq. (9) [15]. In this model, we adopt the transfer function with varying frequency ratio for wind excitation given in units of acceleration, i.e., $J(n) = \frac{1}{\sqrt{[1-(n/n_0)^2]^2+4\xi^2(n/n_0)^2}}$ (n is the frequency of the wind, n_0 is the natural frequency of structures, ξ is the damping ratio) - derived by Martinez-Vazquez [16].

$$\sigma_a^2 = \int_0^\infty |J(n)|^2 S_{cu}(z_i, z_j, n)dn \tag{7}$$

$$\sigma_{a,b}^2 = \int_0^\infty S_{cu}(z_i, z_j, n)dn \tag{8}$$

$$\sigma_{a,r}^2 = S_{cu}(z_i, z_j, n_0)\int_0^\infty |J(n)|^2 dn \cong \frac{\pi n_0 S_{cu}(z_i,z_j,n_0)}{4\xi} \tag{9}$$

Furthermore, the design spectra of the output acceleration can be evaluated by the square root of the sum of the background component (see Eq. (8)) and the resonant component (see Eq. (9)). Therefore, the design spectra for vertical MDOF system are defined as Eq. (10).

$$S_a = \sqrt{\sigma_a^2} = \sqrt{\sigma_{a,b}^2 + \sigma_{a,r}^2} \tag{10}$$

3 ESTIMATION OF WIND DESIGN SPECTRA

The input variables of the wind design spectra are investigated based on the building dimensions with the width of 20m, the aspect ratio of 10, and the ratio between chord and width of 1. The damping ratio of this building is 0.025 and the assumed height of peak velocity of downburst wind is 50m. The reference velocity at 10-metre height above the ground is set as 25m/s. Then, the overall design spectra of the output acceleration with varying $\overline{U}_{max,10}$ is shown in Figure 1 (a) with varying natural period, T. According to the properties of the structures, it is reasonable to assume that the investigating range of the natural frequency changes from 0.1Hz to 10Hz and the corresponding range of the natural period varies from 10s to 0.1s. When the natural frequency of the structure matches 0.2, the overall design spectra of the output acceleration at 50m/s is about 5 times higher than the counterpart at 25m/s. When the natural frequency reduces to 0.1, the ratio of the overall design spectra at 50m/s and 25m/s is around 5.4.

The dimensions of the structure are also non-negligible, shown in Figure 1(b) for the aspect ratio, H/W and Figure 1(c) for the ratio of L/W. The descending dimensions of the structures relative to the structural width can lead to higher design spectra for both ratios of H/W and L/W, since lower structures have higher natural frequency with high acceleration. For the natural frequency of 0.2, the ratio of the design spectra when $H/W = 1$ and $H/W = 10$ is around 4.2. The ratio of these decreases to 4.0 for the natural frequency of 0.1. Similarly, the ratio of the design spectra when $L/W = 0.4$ and $L/W = 2$ is around 5.1 and 5.0 for $n_1 = 0.2$ and $n_1 = 0.1$ respectively. This tendency demonstrates that when the natural periods of the structures increase from 5 to 10, the effects of the input parameters mitigate.

A slight influence of the height of peak velocity can be observed in Figure 1 (d), since the height of peak velocity approaching the top of the building can increase the velocity at the top of the building. Therefore, the reference velocities of the downburst outflow and the dimensions of the structures are important parameters for the design spectra of the output acceleration.

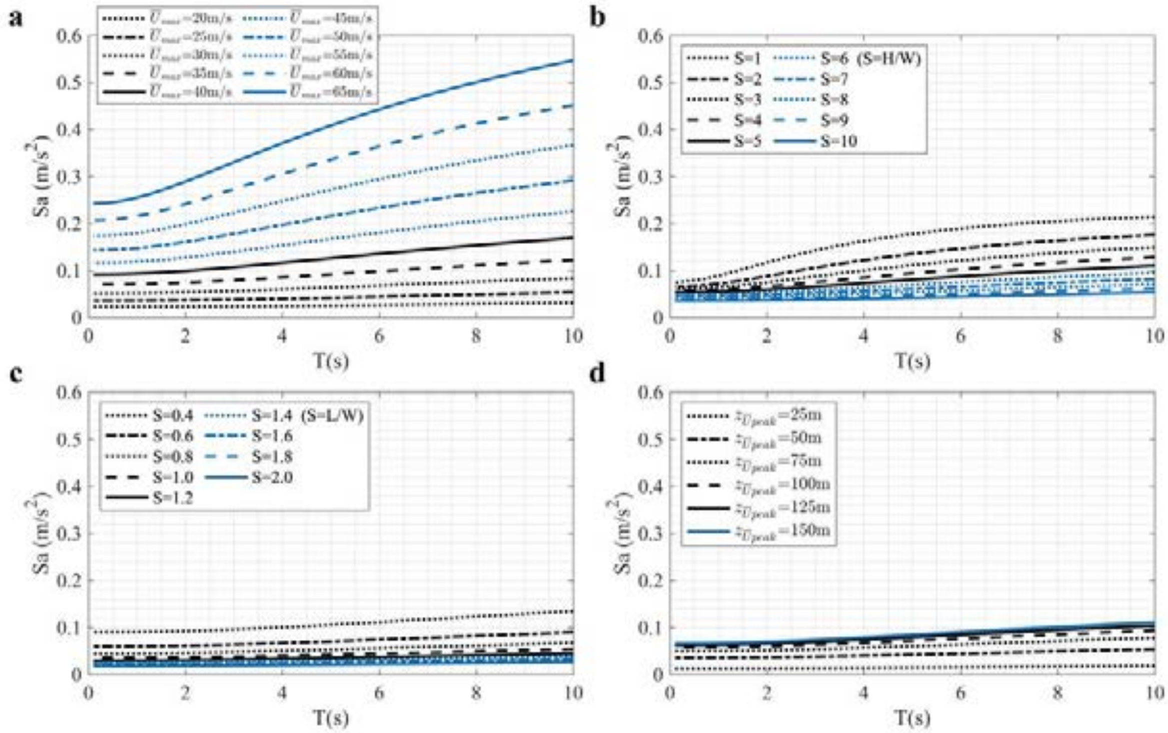

Figure 1: Design spectra of output acceleration with variable parameters, (a) $\overline{U}_{max}$; (b) H/W; (c) L/W; (d) $z_{\overline{U}_{peak}}$.

4 DYNAMIC RESPONSE OF CAARC TALL BUILDING

4.1 CAARC tall building

The CAARC benchmark building is utilized in this section for the validation of the wind design spectra. The CAARC building was experimentally analysed in five different laboratories and has been regarded as a standard prototype of the tall building to study on the dynamic analysis [21]. The dimensions of the building are shown in Figure 2. The natural frequency of the building is 0.2Hz along the x and y direction. The fraction of the critical damping equals 0.01 and the mass per unit volume of the building is 160 $kg \cdot m^{-3}$. The turbulence intensity at the top of the building is around 10.57% and a linear relationship of the turbulence intensity with varying height is assumed, where the turbulence intensity at height of 0.5H is around 13.34%.

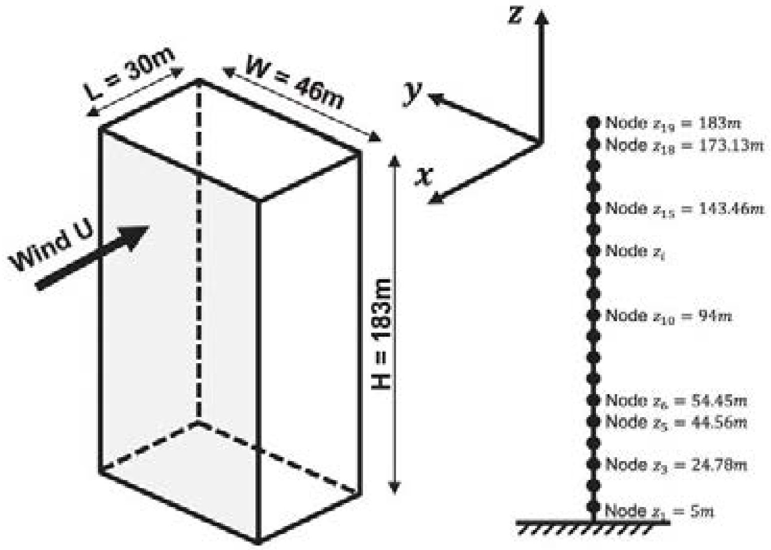

Figure 2: The dimensions of the CAARC benchmark building.

4.2 Application of wind design spectra and validation

The CAARC building was assumed as a uniform cantilever structure with a series of single-degree-of-freedom (SDOF) oscillators. This building is equally divided into 18 elements with 19 nodes in the vertical directions of the domain, i.e., $z_i = (i-1) \times 9.89m, i = 1, 2, \ldots, 19$ starting from 5m to 183m (see Figure 2). In running a modal spectral analysis (where the proposed design spectra fit), the dynamic displacement response, σ_d (rms of dynamic displacement), can be obtained by using Eq. (11). In addition, the static displacement response of the structure can be obtained with Eq. (12) which is the ratio of the first modal static force (Eq. (13)) and the first modal stiffness $k_1 = 4\pi^2 n_1^2 m_1$.

$$\sigma_{1d}(z) = \phi_1(z) \frac{L_1 S_a}{m_1 (2\pi n_1)^2} \tag{11}$$

$$d_{1s}(z) = \frac{\bar{f}_1(z)}{k_1} \tag{12}$$

$$\bar{f}_1(z_i) = c_D \times 0.5 \rho W \bar{U}_{max}^2(z_i) \sum_1^i \left[\int_{z_i}^{z_{i+1}} \alpha^2(z) \phi_1(z) dz \right] \tag{13}$$

In these equations, ϕ_1 is the first modal amplitude at height z and can approximately take the form $\phi_1(z) = (z/H)^{\psi}$ with H representing the vertical dimension of structures. The constant ψ is taken as 1 in this case since a frame structure with large columns and shear bracings is considered [22]. Further variables n_1, m_1 and L_1 represent the first natural frequencies, first modal mass, $m_1 = \int_0^H m_l(z) \phi_1^2(z) dz$ (where m_l is the structural mass per unit height), and first modal excited masses, $L_1 = \int_0^H m_l(z) \phi_1(z) dz$), respectively. In Eq. (13), i represents the i-th number of recorded velocity time history and $\alpha(z) = \bar{U}_{max}(z)/\bar{U}_{max}(z_i)$ which is the non-dimensional vertical shape profile of horizontal wind. Finally, the peak displacement response can be obtained by $d_{1T}(z) = d_{1s}(z) + \sigma_{1d}(z)$.

The displacements at the top of the building calculated from the wind design spectra are shown in Table 1. In Eq. (4), the correlation function in both vertical and transverse directions of the windward surface of the building is considered. The results from the design spectra combined with the partially correlated function only in vertical direction are also demonstrated in Table 1, where the oscillation amplitude of the displacement at the top of the building in x direction, i.e., 0.351m, is larger than 0.328m since the correlation in the transverse direction is considered as full correlation, that is the same consequences as the displacement response in y-direction where the counterpart is 0.206m which is higher than 0.196m. Higher correlation can cause higher displacement amplitude of the oscillation. The peak displacements calculated from the simulated wind field using time series analysis methods, i.e., the Newmark method (M1) and interpolation of excitation method (IOE, M2) (detailed in Chopra [23]), are illustrated in Table 1, where the results are slightly higher than those from the wind design spectra.

Direction	Design spectra for thunderstorm downburst					Num. Int. (M1)	Num. Int. (M2)
		Davenport's coherence (vertical and horizontal)		Partial correlated (vertical only)		Partial correlated (vertical only)	
	Static	Dynamic	Total	Dynamic	Total	Total	Total
x	0.200	0.128	0.328	0.151	0.351	0.435	0.434
y	0.112	0.084	0.196	0.094	0.206	0.244	0.243

Table 1: Lateral displacements at the top of the building (Unit: metre).

5 CONCLUSIONS

This paper presents a newly developed design spectra technique in the frequency domain subjected to thunderstorm downburst which attempts to extend the successful performance-based design philosophy to wind engineering application. The investigation for the input parameters of the design spectra illustrates that the design spectra of the output accelerations significantly rise with the increasing velocities of the downburst outflow, and obviously mount with the decreasing aspect ratio and cross-sectional ratio of the buildings. The consistent comparison between the results of the wind design spectra in frequency domain and the equivalent numerical simulation in time domain for the benchmark tall building enable design spectra technique to be developed, assuming that the displacement response is mainly reflected on the first mode. This technique identifies the mechanical properties of the structure and aerodynamic characteristics of the outflow winds, which enable to obtain the structural performance for MDOF oscillators. Further validations of the proposed design spectra by using the finite element method will be carried out.

REFERENCES

[1] J. R. McDonald and K. C. Mehta, *A recommendation for an Enhanced Fujita scale (EF-Scale)*. Wind Science and Engineering Center, Texas Tech University, 2006.

[2] T. T. Fujita, "The downburst: microburst and macroburst. University of Chicago," *SMRP Research* vol. 210, p. 112, 1985.

[3] T. T. Fujita, "Downbursts: meteorological features and wind field characteristics," *Journal of wind engineering and industrial aerodynamics,* vol. 36, pp. 75-86, 1990.

[4] M. Chay and F. Albermani, "Dynamic response of a SDOF system subjected to simulated downburst wind," 2005.

[5] L. Chen and C. Letchford, "A deterministic–stochastic hybrid model of downbursts and its impact on a cantilevered structure," *Engineering structures,* vol. 26, no. 5, pp. 619-629, 2004.

[6] X. Chen, "Analysis of alongwind tall building response to transient nonstationary winds," *Journal of structural engineering,* vol. 134, no. 5, pp. 782-791, 2008.
[7] E. C. Choi and F. A. Hidayat, "Dynamic response of structures to thunderstorm winds," *Progress in Structural Engineering and Materials,* vol. 4, no. 4, pp. 408-416, 2002.
[8] J. Holmes, G. Forristall, and J. McConochie, "Dynamic response of structures to thunderstorm winds," in *Proc., 10th Americas Conf. on Wind Eng.(10ACWE)*, 2005, pp. 1-6.
[9] G. Solari, "Thunderstorm response spectrum technique: theory and applications," *Engineering Structures,* vol. 108, pp. 28-46, 2016.
[10] G. Solari, D. Rainisio, and P. De Gaetano, "Hybrid simulation of thunderstorm outflows and wind-excited response of structures," *Meccanica,* vol. 52, no. 13, pp. 3197-3220, 2017.
[11] K. A. Skalomenos, G. D. Hatzigeorgiou, and D. E. Beskos, "Application of the hybrid force/displacement (HFD) seismic design method to composite steel/concrete plane frames," *Journal of Constructional Steel Research,* vol. 115, pp. 179-190, 2015.
[12] D. N. Serras, K. A. Skalomenos, and G. D. Hatzigeorgiou, "A displacement/damage controlled seismic design method for MRFs with concrete-filled steel tubular columns and composite beams," *Soil Dynamics and Earthquake Engineering,* vol. 143, p. 106608, 2021.
[13] G. W. Housner, R. Martel, and J. Alford, "Spectrum analysis of strong-motion earthquakes," *Bulletin of the Seismological Society of America,* vol. 43, no. 2, pp. 97-119, 1953.
[14] G. Solari, "Wind response spectrum," *Journal of Engineering Mechanics,* vol. 115, no. 9, pp. 2057-2073, 1989.
[15] E. Simiu and R. H. Scanlan, "Wind effects on structures: fundamentals and applications to design," 1996.
[16] P. Martinez-Vazquez, "Wind-induced vibrations of structures using design spectra," *International Journal of Advanced Structural Engineering,* vol. 8, no. 4, pp. 379-389, 2016.
[17] P. Martinez-Vazquez, "Wind design spectra for generalisation," *Wind and Structures,* vol. 30, no. 2, p. 155, 2020.
[18] G. Solari, M. Burlando, P. De Gaetano, and M. Repetto, "Characteristics of thunderstorms relevant to the wind loading of structures," *Wind and Structures,* vol. 20, no. 6, p. 763, 2015.
[19] A. Davenport, "The prediction of the response of structures to gusty wind," *Safety of structures under dynamic loading,* vol. 1, pp. 257-284, 1977.
[20] G. S. Wood, K. C. Kwok, N. A. Motteram, and D. F. Fletcher, "Physical and numerical modelling of thunderstorm downbursts," *Journal of Wind Engineering and Industrial Aerodynamics,* vol. 89, no. 6, pp. 535-552, 2001.
[21] W. Melbourne, "Comparison of measurements on the CAARC standard tall building model in simulated model wind flows," *Journal of Wind Engineering and Industrial Aerodynamics,* vol. 6, no. 1-2, pp. 73-88, 1980.
[22] C. Eurocode, "1: Actions on structures–Part 1.4: General actions–wind actions," *Brussels: The European Standard EN,* 2010.
[23] A. K. Chopra, *Dynamics of structures : theory and applications to earthquake engineering*, 4th ed. ed. Upper Saddle River, N.J. : Prentice Hall ; London : Pearson Education [distributor], 2011 (in English), 2011.