Publishers

Published by Avanti Publishers

International Journal of Architectural Engineering Technology

ISSN (online): 2409-9821

Estimation of Wind Effects on High-Rise Structures by the Global Load Effects and Database-Assisted Design Methods

Emil Simiu*

Emil Simiu Consulting, N. Bethesda, MD, USA

ARTICLE INFO

Article Type: Research Article

Academic Editor: Eric M. Lui

Keywords:
Risk assessment
Wind engineering
High-rise buildings
Global load effects
Property insurance
Structural engineering
Database-assisted design
Demand-to-capacity indexes

Timeline:
Received: February 29, 2025
Accepted: April 11, 2025
Published: May 05, 2025

Citation: Simiu E. Estimation of wind effects on high-rise structures by the global load effects and database-assisted design methods. Int J Archit Eng Technol. 2025; 12: 1-7.

DOI: https://doi.org/10.15377/2409-9821.2025.12.1

ABSTRACT

Structural wind effects on high-rise buildings subjected to extreme directional winds can be determined by one of three methods: (i) HFB (high frequency balance), used only for buildings with very complex shapes or with many fine-scale features, (ii) GLE (global load effects), commonly used in current commercial wind engineering laboratory practice, and (iii) the DAD (Database Assisted Design) method. The purpose of this paper is to consider the advantages and drawbacks of the GLE and DAD methods, both of which use the multi-channel pressure scanning system. Following these methods' brief description, it is noted that the GLE method has over DAD the advantage of significantly lower computational time requirements. This is shown to be due to GLE's basic assumption that the peak Demand-to-Capacity Indexes (DCIs) of all the building's structural members occur at the same time. It is then shown that this assumption is incorrect, and that it results in the GLE method's underestimation of all the structural members' DCIs, inter-story drift ratios, and top floor accelerations. In contrast, the DAD method is shown to satisfy all applicable strength and serviceability performance criteria. However, the computational resources required for DAD's use exceed the resources typically available to small or mid-sized structural design offices. Recent research results concerning the DAD method are then noted, and various approaches are proposed to the reduction of the DAD method's computation time requirements by up to two orders of magnitude. It is then suggested that the potential of the DAD method as a risk assessment tool for buildings designed by the GLE method could be considered for property insurance purposes.

*Corresponding Author
Email: simiue@aol.com

1. Introduction

The High-Frequency Balance (HFB) method, widely used before the development of the multi-channel pressure scanning system, is currently employed only for "buildings with very complex shapes or with many fine-scale features, such as lattices, for which it is not possible to install pressure taps in all the required locations to accurately resolve the overall forces on the building" [1, p. 22]. Also, because it provides no reliable information on aerodynamic torques, the HFB method should not be used if the structural effects induced by the aerodynamic torsional moments are significant. For these reasons structural wind effects on most high-rise buildings subjected to extreme directional winds can be determined by using either the global load effects (GLE) [1, 2] or the Database Assisted Design (DAD) method [3-8].

The purpose of this article is to consider these methods' advantages and drawbacks. Both methods are briefly described, and it is noted that the GLE method has over DAD the advantage of significantly lower computation time requirements. This is shown to be due to the assumption inherent in the GLE method that the peak Demand-to-Capacity Indexes (DCIs) of all the building's structural members occur at the same time. It is then shown that this assumption is incorrect, and that it results in the GLE method's underestimation of all structural members' DCIs, inter-story drift ratios, and top floor accelerations. In contrast, the DAD method is shown to satisfy all applicable strength and serviceability performance criteria. However, the computational resources required for DAD's use exceed the resources typically available to small or mid-sized structural design offices. Recent research results concerning the DAD method are then noted, and various approaches are proposed to the reduction of the DAD method's computation time requirements by up to two orders of magnitude. It is then suggested that the potential of the DAD method as a risk assessment tool for buildings designed by the GLE method could be considered for property insurance purposes.

Both GLE and DAD assume linearly elastic material behavior. The structural performance of a 47-story existing building subjected to extreme wind loads that induced non-linear material behavior was recently assessed in [11]. However, to the author's knowledge, high-rise structures subjected to extreme wind loading are still typically designed by assuming linearly elastic response. This is the case for both GLE and DAD.

The material presented herein concerns the following topics: Wind engineering tasks in support of both GLE and DAD; Wind engineering tasks specific to GLE; DAD: Structural engineering tasks; DAD: Demand-to-capacity indexes; *N*-year peak demand-to-capacity indexes; Recent research results and suggested future research. The final section presents the conclusions of this work.

2. Wind Engineering Tasks in Support of both GLE and DAD

The following tasks are completed by the wind engineering laboratory in support of both GLE and DAD:

Task 1. Description of the micrometeorological features of the building site. This task consists of determining the terrain roughness length as a function of wind direction, and enables the adequate wind tunnel simulation of directional atmospheric flows.

Task 2. Development of a sample of extreme directional wind speeds at the building site, $V_n(h, \theta_k)$, based on measured or simulated historical extreme wind speed data at a meteorological site with standard open terrain roughness (n = 1, 2,..., n_{max}, n_{max} is a sufficiently large sample size, h = height above ground, e.g. $h = 10$ m, and θ_k = wind direction, k = 1, 2,.., k_{max}, e.g., θ_k = 22.5, 45,..., 360°).

Task 3. Simultaneous measurement of statistically stationary time histories of aerodynamic pressure $p_j[V(h, \theta_k),t)$ ($0 < t < t_{max}$) at a sufficient number of pressure taps (j = 1, 2,.., j_{max}) placed on the exterior surfaces of the wind tunnel building model. This task enables the determination of the requisite time-dependent aerodynamic pressure coefficients.

Task 4. Use of time histories of measured aerodynamic pressures to produce time histories of mutually orthogonal aerodynamic force components and torsional moments acting at each floor's center of mass, and determining the respective non-dimensional force and torque coefficients.

3. Wind Engineering Tasks Specific to GLE

Task 1. Using data on the structure's modes of vibration, natural frequencies and damping ratios provided by the structural engineer, and the aerodynamic data obtained in Task 4 of the preceding section, perform dynamic analyses resulting in time series of inertial forces and torque acting at each floor, and time series of the effective (aerodynamic + inertial) wind-induced mutually orthogonal forces and torques applied at the center of mass O_i of each floor i (i = 1, 2,.., i_{max}) for each wind velocity $V(h, \theta_k)$). These time series are denoted by $F_{ix,w}(V(h), \theta_k, t)$, $F_{iy,w}(V(h), \theta_k, t)$, $T_{i,w}(V(h), \theta_k, t)$, where the index w designates values obtained in current practice by the wind engineering laboratory.

Task 2. Determining, and delivering to the structural engineer, the times $t_{max}[(V(h), \theta_k)]$ at which the absolute value of the combined base moments attains its peak. The expressions for the base moments are:

$$M_{by}(V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} F_{ix,w}(V(h), \theta_k, t)\, H_i \tag{1a}$$

$$M_{bx}(V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} F_{iy,w}(V(h), \theta_k, t)\, H_i \tag{1b}$$

$$T_b(V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} T_{i,w}(V(h), \theta_k, t) \tag{1c}$$

where H_i is the height above ground of floor i.

It is stated in [1, p. 26] that "In structural design the important variables for the designer are typically load effects such as the base bending moments, base shear, base torsion and corresponding force and torque distribution with height. These global load effects are selected because they are closely correlated with the load levels reached in individual structural members…" (Note: The term "load effects" as used in [1] refers to base moments, shears and torques, i.e., to resultant *loads*. To avoid possible confusion, it is appropriate to reserve the term "load effects" to the structural effects of the loads, rather than to resultant loads).

Inherent in the statement just quoted is the GLE basic assumption that all the peak wind effects of interest induced in the structure by wind with velocity $V(h, \theta_k)$ occur at the same time, denoted by $t_{max}(V(h, \theta_k))$, at which the base moment attains its peaks. This assumption was also adopted in [2]. Its advantage is that it simplifies the design process. However, as shown in Section 5, it has the major drawback of underestimating every peak wind effect of interest, including all the members' peak DCIs, by amounts that can be substantial, as shown in Section 5. The elimination of this drawback is the raison d'être of the database-assisted design approach.

4. DAD: Structural Engineering Tasks

1. Perform a preliminary design of the structure using (i) simplified design procedures specified by the applicable building code and (ii) the extreme wind speeds sample $\max_k[V_n(h, \theta_k)]$ (n = 1, 2,…., n_{max}) obtained from the wind velocity data provided by the wind engineering laboratory.

2. Perform dynamic analyses of the structure subjected to the aerodynamic forces and torques induced at each building floor by a sufficient number of wind velocities $V_q(h, \theta_k)$ (q = 1, 2,.., q_{max}). In view of the superior dynamic analysis capabilities of the structural engineering office, this task is assigned in the DAD method to the structural engineer. The velocities $V_q(h, \theta_k)$ need to cover the range of possible extreme wind speeds at the building site and the range of wind directions of interest. The dynamic analyses result in time series of the effective (aerodynamic + inertial) force components $F_{i,ef\,x}[V_q(h, \theta_k)]$, $F_{i,ef\,y}[V_q(h, \theta_k)]$ and torques $T_{i,ef}(V_q(h, \theta_k, t)]$ acting at the centers of mass O_i of floors i.

3. Determine the peak structural responses (e.g., the peak DCIs induced in each structural member m of interest by the effective loads at floors i due to wind with velocities $V_q(h, \theta_k)$.

4. For each velocity $V_q(h, \theta_k)$ estimate the peak structural response with specified mean recurrence interval N.

5. Redesign the structure as needed with a view to satisfying the applicable performance criteria.

5. DAD: Demand-to-Capacity Indexes

In this section it is shown that the time history of the DCI, and therefore its peak and the time of occurrence of the peak, differ from member to member.

As an example of wind effects consistently underestimated by the GLE method, consider the cross section of a steel column, denoted by *m*, of a building subjected to loads induced by wind with velocity $V(h, \theta_k)$. Assume that $P_{rm}/(\Phi_P P_{am}) \geq 0.2$, where P_{rm} = required factored compressive strength, Φ_P = resistance factor, and P_{am} = available compressive strength. For such a column the peak DCI is

$$\mathrm{DCI}_{m\ pk} = \frac{P_{rm}}{\phi_P P_{am}} + \frac{8}{9}\left(\frac{M_{r1m}}{\phi_M M_{a1m}} + \frac{M_{r2m}}{\phi_M M_{a2m}}\right) \tag{1}$$

[12]. The required axial force $P_{rm}(V(h), \theta_k, t)$ and bending moments $M_{r1m}(V(h), \theta_k, t)$,

$$P_{rm}\ (V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} [F_{i,efx}(V(h), \theta_k, t)\, p_{x\,im} + F_{i,efy}(V(h), \theta_k, t)\, p_{y\,im} + T_{i,ef}(V(h), \theta_k, t)\, p_{T\,im}] + P_{gm}$$

$$M_{r2m}\ (V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} [F_{i,efx}(V(h), \theta_k, t)\, \mu_{1x\,im} + F_{i,efy}(V(h), \theta_k, t)\, \mu_{1y\,im} + T_{i,ef}(V(h), \theta_k, t)\, \mu_{1T\,im}] + M_{1gm}$$

$$M_{r2m}\ (V(h), \theta_k, t) = \sum_{i=1}^{i_{max}} [F_{i,efx}(V(h), \theta_k, t)\, \mu_{2x\,im} + F_{i,efy}(V(h), \theta_k, t)\, \mu_{2y\,im} + T_{i,ef}(V(h), \theta_k, t)\, \mu_{2T\,im}] + M_{2gm} \tag{2a, b, c}$$

where P_{gm}, M_{1gm}, M_{2gm} denote the internal forces induced by the gravity loads in cross section *m*, and the constants $p_{x_{im}}, p_{y_{im}}, p_{T_{im}}, \mu_{1_m}\ \mu_{1y_{im}}, \mu_{1T_{im}}, \mu_{2x_{im}}, \mu_{2y_{im}}, \mu_{2T_{im}}$ are influence coefficients (e.g., $p_{x_{im}}$ is the axial force induced in cross section *m* by a unit force in the *x* direction acting at the center of mass of floor *i*). After some algebra it follows that

$$\mathrm{DCI}_m\,[V(h), \theta_k, t] = \sum_{i=1}^{i_{max}} [C_{F\,ix,m}\, F_{i,efx}(V(h), \theta_k, t) + C_{F\,iy,m}\, F_{i,efy}(V(h), \theta_k, t) + C_{T\,im}\, T_{i,ef}(V(h), \theta_k, t)] + C_{gm} \tag{3}$$

where the quantities $C_{F_{ix,m}}, C_{F_{iy,m}}, C_{F_{ix,m}}$ are functions of properties of the structure, i.e., of available strengths, resistance factors, and influence coefficients (see [9] for details).

Since influence coefficients differ from member to member, so do the respective DCI_m time series and the times of occurrence of their peaks, $t^m_{max}(V(h), \theta_k)$, where the index *m* identifies the structural member. This fact invalidates the GLE assumption that the peak demand-to-capacity occurs for all members at the same time, t_{max} $(V(h), \theta_k)$. Since, for any velocity $V(h)$, θ_k, $\mathrm{DCI}_m(t_{max}) < \mathrm{DCI}_m(t^m_{max})$ almost surely, it follows that this assumption results in the underestimation of the wind effect being considered. The magnitude of the underestimation varies randomly within the open interval (0, $\max_t\{\mathrm{DCI}_m[(V(h), \theta_k, t]\} - \min_t\{\mathrm{DCI}_m[(V(h), \theta_k, t]\}$) (there would be no underestimation if $t_{max} = t^m_{max}$, i.e., if the peak base moment occurred at the same time as the peak DC I; and the underestimation would be largest if $t_{max} = t^m_{min}$, where t^m_{min} denotes the time of occurrence of the lowest value of the DCI, $\min_t\{\mathrm{DCI}_m[(V(h), \theta_k, t]\}$). Similar statements apply to top floor accelerations and inter-story drift ratios.

6. *N*-Year Peak Demand-to-Capacity Indexes

The entries in the matrix with n_{max} rows and k_{max} columns

$$[\mathrm{DCI}_m\{(V_n(h), \theta_k)\ , t^m_{max}(V_n(h), \theta_k)\}] \tag{4}$$

are peak DCIs determined by the DAD method, induced in member *m* (m = 1, 2..., $m_{\max}$) by wind with extreme velocities at the building site $V_n\ (h)$, θ_k (n = 1, 2,..., $n_{\max}$; k = 1, 2,...,k_{max}) at member-dependent times $t^m_{max}(V_n(h), \theta_k)$, at which the time-dependent demand-to-capacity DCIs attain their peaks [10]. To estimate peak DCIs with any specified *N*-year return period, a vector is created, the n_{max} components of which are the largest entry in each matrix row, $max_k[\mathrm{DCI}_m\{(V_n(h), \theta_k)\ , t^m_{max}(V_n(h), \theta_k)\}]$. A univariate probability distribution of the largest values is then fitted to those components. If the *N*-year peak DCI of member *m* exceeds unity or is significantly lower than unity, that member needs to be redesigned. This approach supersedes the approach proposed in [13], which is based on peak wind speeds over periods of time far shorter that one year and therefore violates the basic assumption underlying extreme value estimation theory.

The process by which the GLE estimate of the *N*-year peak DCI is performed is similar to the process just described. It involves a matrix similar to Eq. 4. However, since every entry in the GLE matrix is lower than its counterpart determined by the DAD method, the GLE method underestimates the *N*-year peak DCIs of all the structure's members.

7. Recent Research Results and Suggested Future Research

Software for the implementation of the DAD method was developed for prismatic steel structures with a rectangular shape in plan, using MATLAB and ETABS software [7]. The DAD method was recently expanded for use on reinforced concrete members and irregular-shaped buildings, and the MATLAB software was replaced by Python, thus making the software more accessible to structural engineering offices [6].

Computation times have been substantially reduced through the use of parallel computing, the simultaneous running of ETABS models, and the restructuring of algorithms resulting in more efficient, matrix-based computations [6]. Further reduction of computation times remains an important objective. The following are potential means of achieving such reduction by up to two orders of magnitude:

1. The length of the stationary portion of the time histories of the wind effects currently assumed in North American practice is 60 min. That length could be reduced to 10 min, as assumed in current Japanese practice. This would reduce the precision of the estimated DCI peak and would therefore result in a typically minor increase of the wind load factor. A generic study of that increase could be performed by transforming a typical non-Gaussian DCI time series into a Gaussian time series, and applying the relationship between Gaussian time series length and peak statistics developed in [14] or [15]. The effect of the larger uncertainty in the peak value on the magnitude of the wind load factor would then be readily estimated by using the approach presented in [10, Ch. 7].

2. For any given wind direction, previous DAD research on peak DCIs has considered 20, 30, 40, 50, 60, 70, and 80 m/s wind speeds. Currently, peak DCIs are determined for each of those speeds by performing the computer-intensive operations described earlier herein. It is suggested that the feasibility be considered of performing those operations for, e.g., only the 20, 50, and 80 m/s speeds, and of accurately determining the DCIs for the 30, 40, 60, and 70 m/s by nonlinear interpolation.

3. Rather than computing the wind effects induced by the effective wind loads acting at each building floor i (i = 1, 2, ...i_{max}), the possibility could be examined of computing the wind effects induced by the resultants of the aerodynamic loads acting at more than one floor, e.g. at three consecutive floors, with those resultants acting at the center of mass of the second floor in each of the three-floor groups.

4. The multiple-points-in-time approach developed in [16] was shown to produce remarkably accurate estimates of peak combined wind effects by using a limited number of peaks from the time histories of the individual wind effects being combined. Those estimates are obtained far more economically in terms of computational time than conventional time domain estimates that use full time histories.

In addition, it is suggested that the potential of the DAD method as a risk assessment tool for buildings designed by the GLE method may be considered for property insurance purposes.

8. Conclusions

It has been shown that the GLE method, currently widely used by wind engineering laboratories, underestimates wind effects on high-rise structures. The underestimation can be expected to be substantial for large numbers of structural members, top floor accelerations and inter-story drift ratios, and therefore to result in structural designs that do not satisfy safety and serviceability performance requirements.

Earthquake engineering has historically benefited from intense study in academic and research institutions. In contrast, wind engineering as applied to structural design has been the object of study in relatively few institutions. This may explain the inadequate scrutiny of the GLE method by both wind and structural engineering practitioners.

A review of a recent manual on the performance of tall buildings under wind loads [17] confirms the weakness of that scrutiny.

The present work suggests that the wind engineering laboratory needs to limit its products to those listed in Section 2. The structural engineer's tasks then include performing the requisite structural dynamics analyses and determining the requisite peak DCIs, top floor accelerations and inter-story drift ratios, with return periods specified by the respective performance requirements. These tasks are performed effectively by using DAD. The GLE method results in unconservative designs because base moments are not an acceptable substitute for the large numbers of structural data, including influence coefficients, on which the DCIs are dependent and to which the wind engineering laboratory cannot in practice have access.

Conflict of Interest

The author has no known competing financial interests or personal relationships that could have appeared to influence the work reported herein.

Funding

This work was performed with no funding from any fund granting body.

Acknowledgments

The author wishes to thank the anonymous reviewers for comments that helped to improve this work, and Dr. Daniel Rhee of the Structures Group, National Institute of Standards and Technology (NIST), for providing useful information on recent NIST Database-Assisted Design research.

References

[1] Irwin PA, Denoon R, Scott D. Wind tunnel testing of high-rise buildings. Abingdon: Routledge; 2013. https://doi.org/10.4324/9781315879529

[2] Boggs D, Lepage A. Wind tunnel methods. 2008. Available from: https://cppwind.com (accessed on 15 April 2025).

[3] American Society of Civil Engineers. Minimum design loads and associated criteria for buildings and other structures (ASCE 7-22). Reston (VA): ASCE; 2022.

[4] Merhi A, Letchford CW. Computational method in database-assisted design for wind engineering with varying performance objectives. Wind Struct. 2021; 32(5): 439-52.

[5] Park S, Yeo D. Introductory tutorial for DAD: design examples of high-rise buildings for wind. NIST Tech Note 2001. Gaithersburg (MD): NIST; 2018. https://doi.org/10.6028/NIST.TN.2001

[6] Rhee D, Yeo D, Mashnad M, Carman B, Clay C. Database-assisted design approach for dynamically sensitive steel and reinforced concrete structures. NIST Tech Note 2308. 2024. https://doi.org/10.6028/NIST.TN.2308

[7] Saini D, Alipour A, Shafei B. Database-assisted design of high-rise steel structures using ETABS software. User's Manual, DAD-ETABS version 1. 2022. https://doi.org/10.6028/NIST.GCR.22-035

[8] Saini D, Dokhaei B, Shafei B, Alipour A. Performance evaluation of high-rise buildings using the database-assisted design approach. Struct Saf. 2024; (*in press*). https://doi.org/10.1016/j.strusafe.2024.102447

[9] Saini D, Heckert NA, Simiu E. N-year demand-to-capacity indexes and the design of high-rise buildings for wind. Structures. 2024; https://doi.org/10.1016/j.istruc.2024.106488

[10] Simiu E, Yeo D. Wind effects on structures. 4th ed. Oxford: Wiley Blackwell; 2019. https://doi.org/10.1002/9781119375890

[11] Mohammadi A, Azizinamini A, Griffis A, Irwin P. Performance assessment of an existing 47-story high-rise building under extreme wind loads. J Struct Eng. 2019; 145(1): 04018232-1. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002239

[12] American Institute for Steel Construction. Specifications for structural steel buildings AISC 360-10. Chicago (IL): AISC; 2010.

[13] Isyumov N, Ho E, Case P. Influence of wind directionality on wind loads and responses. J Wind Eng Ind Aerod. 2014; 114: 169-80. https://doi.org/10.1016/j.jweia.2014.06.006

[14] Davenport AG. Note on the distribution of the largest value of a random function with application to gust loading. Proc Inst Civ Eng. 1964; 28(2): 187-96. https://doi.org/10.1680/iicep.1964.10112

[15] Preumont A. On the peak factor of stationary Gaussian processes. J Sound Vib. 1985; 100: 15-34. https://doi.org/10.1016/0022-460X(85)90339-6

[16] Yeo D. Multiple points-in-time estimation of peak wind effects on structures. J Struct Eng. 2013; 139(3): 462-71. https://doi.org/10.1061/(ASCE)ST.1943-541X.0000649

[17] Biswas P, Peronto J, editors. Design and performance of tall buildings for wind. ASCE Manuals and Reports on Engineering Practice No. 143. Reston (VA): ASCE; 2020. https://doi.org/10.1061/9780784415658