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Tall Building Database-assisted Design: a Review of NIST Research

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Abstract

The purpose of this review paper is to briefly describe main the features of novel procedures developed by the National of Standards and Technology (NIST) for the design of tall buildings. Topics considered in the paper include: the division of tasks between wind and structural engineers; the determination of wind effects with specified mean recurrence intervals by accounting for wind directionality; the risk-consistent design of structures subjected to multiple wind hazards; iterative dynamic analyses and member sizing, including the use of modern optimization approaches; and commonalities of and differences between Database-assisted Design (DAD) and Equivalent Static Wind Loads procedures. An example of the application of the DAD procedure is presented for a reinforced concrete structure. Also included in the paper is an introduction to ongoing research on the estimation of wind load factors or of augmented design mean recurrence intervals commensurate with the uncertainties in the factors that determine the wind effects.

Keywords: Augmented mean recurrence intervals, Database-assisted design, Equivalent static wind loads, Tall buildings, Wind loads

1. Introduction

Large discrepancies between independent wind engineering laboratories' estimates of wind effects on tall structures, the lack of success of efforts to trace unambiguously the sources of those discrepancies, and the fact that "wind engineering is an emerging technology with no consensus on certain aspects of current practice" (SOM 2004), have prompted the National Institute of Standards and Technology (NIST) to develop a research program on the design of tall structures for wind loads. The research was aimed to: improve the accuracy of estimates of wind effects with specified mean recurrence intervals (MRIs) by solving in a physically and probabilistically rigorous manner outstanding problems associated with the effects of wind directionality; achieve a more effective integration of wind and structural engineers' contributions to the design process; enable the structural engineer to be fully in charge of the structural design process, including the dynamic analyses, heretofore split between wind and structural engineers; and ensure that each phase of the wind and structural engineers' contributions to the structural design is compatible with Building Information Modeling (BIM) requirements and can be effectively scrutinized by structural engineers and project stakeholders.

The NIST research resulted in the development of the procedure called Database-Assisted Design (DAD), and of a variant of the DAD procedure called Equivalent Static Wind Loads (ESWL). In the ESWL approach, which is only applicable to buildings with simple shapes (e.g., prismatic buildings), static wind loads are calculated that induce in the structure demand-to-capacity indexes (DCIs) approximately equal to the more accurate peak DCIs induced by the fluctuating loads (Park et al. 2019). The clarity of both procedures enables their effective scrutiny by structural engineers and project stakeholders, and assures their compatibility with the BIM environment for automated structural design (Borrmann et al. 2018).

The purpose of this review paper is to briefly describe main features of these procedures and provide an example of their application. The following topics are considered herein: the division of tasks between wind and structural engineers in the design process; determining wind effects with specified mean recurrence intervals by accounting for wind directionality; risk-consistent design for multiple wind hazards; member sizing; commonalities of and differences between DAD and EWSL and the potential for using optimization algorithms aimed to reduce material consumption while satisfying strength and serviceability performance criteria. An example of the application of the DAD procedure is presented for a reinforced concrete building. Also included in this work is brief introduction to ongoing research on the estimation of wind load factors and augmented design mean recurrence intervals commensurate with the uncertainties in the factors that determine the wind effects.

For detailed descriptions of the DAD and ESWL procedures and the attendant software, see Park and Yeo (2018a, 2018b, 2018c), Park et al. (2019), and Simiu and Yeo (2019). The software DAD_ESWL is available on www.nist.gov/wind.

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2. Wind and structural engineers' contributions to the design process

The structural design of tall structures for wind has been transformed by the development of (i) the multi-channel pressure scanner, which allows the simultaneous measurement of pressures at multiple taps, and (ii) the capability to store and process vast amounts of data ("big data"). Prior to these developments the design of tall buildings was typically performed by using the High Frequency Force Balance (HFFB). HFFB provides approximate estimates of aerodynamic and dynamic effects derived directly from measurements of base moments and shears. The HFFB procedure provides incomplete information on: the distribution of the wind loads with height (Chen and Kareem 2005); torsional effects; effects associated with fundamental modal shapes other than linear (corrections for nonlinearity being dependent upon the unknown aerodynamic load distribution); and effects of higher modes of vibration. Nevertheless, HFFB remains a useful tool for supporting decisions on building orientation and the improvement of aerodynamic performance, especially in preliminary design stages. One of the features of HFFB is that, once the fundamental dynamic properties of the structure are provided by the structural engineer, the dynamic analysis, which is in large part implicit in HFFB aerodynamic results, is performed by the wind engineer (see e.g., Section 5.6 of Simiu and Yeo 2019).

The current capabilities to measure simultaneously pressures at multiple taps and to store and process "big data" have rendered obsolete the HFFB procedure and the splitting of dynamic analysis tasks between wind and structural engineers. The availability of pressure time histories allows the automatic determination of the aerodynamic loads of interest and, therefore, enables the structural engineering office to perform the full dynamic analysis, a task for which it is typically better equipped than the wind tunnel laboratory.

The tasks of the wind engineer therefore consist of providing the requisite aerodynamics data (i.e., the pressure tap locations and the respective pressure time series), the wind climatological information (i.e., samples of directional wind speeds at the building site's reference height, averaged over one hour or 10 min and obtained from measured or simulated wind speed data), and estimates of uncertainty in the aerodynamic and wind climatological data, required for developing wind load factors or augmented design mean recurrence intervals appropriate for the structure being designed, as indicated in a subsequent section. Once this information is provided by the wind engineering laboratory, the structural engineer fully controls the design process. This division of tasks parallels the division of tasks inherent in design for seismic loads.

3. Wind directionality and wind effects with specified mean recurrence intervals

The sector-by-sector approach to accounting for directional wind effects has been rated as inadequate from a probabilistic point of view, while the outcrossing approach has been criticized for being opaque and therefore impractical (SOM 2004). For these reasons, they were not used in the DAD and ESWL procedures.

The demand-to-capacity index (DCI) is defined as the left-hand side of the design interaction equations used for the sizing of members subjected to more than one type of internal force (e.g., to a bending moment and an axial force); for a well-designed member the DCI is close to unity, subject to serviceability constraints. Consider, for example, a given member cross section for which it is required to determine the peak DCI with a specified design MRI N. To do so it is necessary, as a first step, to develop that cross section's peak DCI *response surface*, that is, the surface representing the peak DCI corresponding to any wind speed and direction. An example of a peak DCI response surface is shown in Fig. 1. It follows from its definition that the response surface is a property of the structure independent of the wind climate. The development of the response surface entails, for each wind speed and direction: using the aerodynamic database provided by the wind engineering laboratory to determine the time series of the aerodynamic loads; performing dynamic analyses to determine the inertial loading; performing structural analyses to determine the DCI time series due to the wind-induced effective (aerodynamic and inertial) loads acting along each of the two principal axes of the structure and in torsion; combining those DCI time series; and estimating the peak of their resultant.

The response surface is used as follows. The wind engineering laboratory provides a climatological directional wind speed matrix $[U_{ij}]$, where i and j identify the storm and the wind direction, respectively; $i = 1, 2,\ldots, i_{max}$; $j = 1, 2,\ldots, j_{max}$. To each speed U_{ij} there corresponds in the response surface a peak DCI_{ij}^{pk}. A new matrix, $[DCI_{ij}^{pk}]$, is then created by substituting in the matrix $[U_{ij}]$ the corresponding values DCI_{ij}^{pk}.

For each storm i it is only the largest of the values

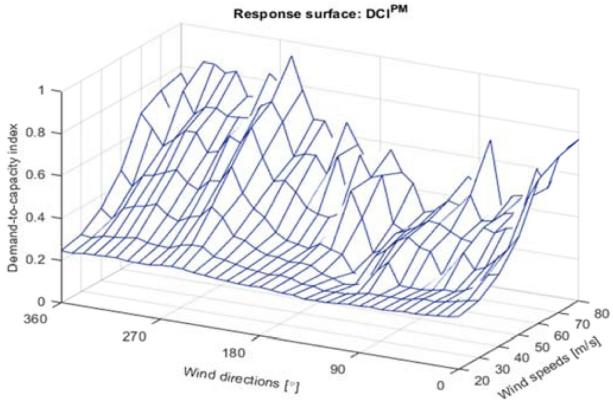


Figure 1. An example: DCI response for a structural member.

DCI_{ij}^{pk}, denoted by $\max_j(DCI_{ij}^{pk})$, that is of interest for structural design purposes. A vector $\max_j(DCI_{ij}^{pk})$ is therefore created, and its components are rank-ordered, thus allowing the use of non-parametric statistics to estimate the peak DCI with the specified mean recurrence interval (MRI) for the cross section being considered. For example, assume that the rank-ordered components of the vector $\max_j(DCI_{ij}^{pk})$ are 0.8, 0.92, 1.01, 1.18, 1.20, 1.25, and that the average interval between storms is one year. The peak DCI corresponding to rank 5 is 1.2, and its MRI is 5 + 1 = 6 years. In practical applications the number of components of the vector $\max_j(DCI_{ij}^{pk})$ must cover a total time interval larger by a factor of 2 or 3, say, than the length of the specified MRI. This criterion determines the number of storm events that must be generated by Monte Carlo simulation. A similar approach is applied in the ESWL procedure. As mentioned earlier, once the aerodynamic and wind climatological data are provided by the wind engineering laboratory, the structural engineer is in charge of all the operations mentioned in this section. All these operations are automated through the use of the DAD_ESWL software.

As explained above, the DAD methodology operates in the space of wind effects for estimating the associated MRIs. This is distinguished from the approach of the ASCE 7-16 Standard which operates in the space of wind speeds for estimating the MRIs of wind effects.

4. Risk consistent-design of multiple wind hazards

Wind hazards at a site can consist of many types of wind storms, such as hurricanes, synoptic winds, thunderstorms and tornadoes. The characteristics of the wind storms can be dissimilar, resulting in different patterns of aerodynamic effects. Because the DAD methodology estimates peak responses with specified MRIs in the space of wind load effects, rather than in the space of wind loads (or wind speeds), DAD can estimate peak joint wind effects from multiple wind hazards by combining peak wind effects from individual wind hazards.

Consider for example a building in a region exposed to both hurricane and synoptic winds (see Sect. 3.1.2 of Simiu and Yeo 2019). Let the random variables R_H and R_S denote the largest yearly hurricane wind effect and the largest yearly synoptic wind effect, respectively. The cumulative distribution function (CDF) of the peak joint wind effect R_{joint} due to both hurricane and synoptic winds can be estimated by

$$P(R_{joint}\le r)=P(R_H\le r \text{ and } R_S\le r)=P(R_H\le r)P(R_S\le r) \tag{1}$$

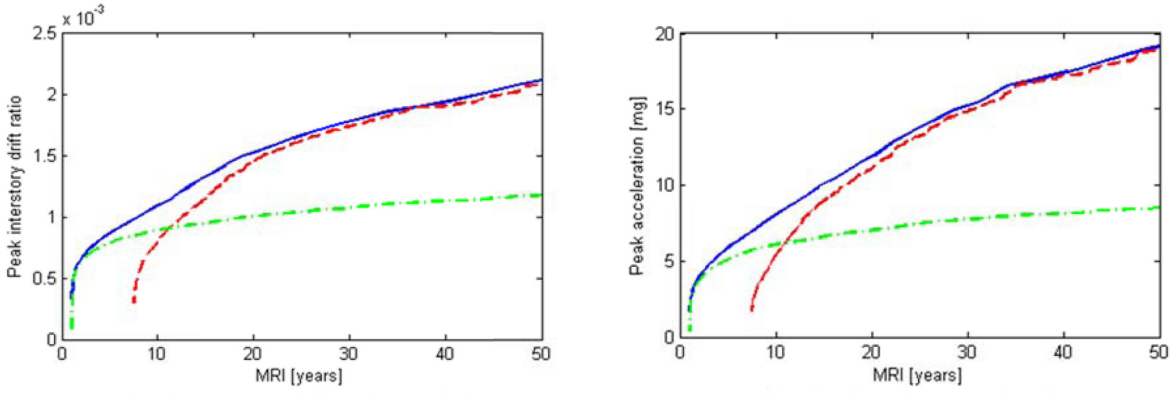
where the variable r denotes a wind effect, and $P(R_H\le r)$ and $P(R_S\le r)$ are the CDFs of the peak wind effect due to hurricane and synoptic winds, respectively.

Using Eq. (1) and the relationship between the CDF and the MRI N (i.e., $P=1-N^{-1}$), the MRI of the peak joint wind effect larger than or equal to r, $N_{\text{joint}}(R_{\text{joint}}\ge r)$, can be estimated as

$$N_{\text{joint}}(R_{\text{joint}}\ge r)=\frac{1}{1-P[R_{\text{joint}}\le r]}$$

$$=\frac{N_H N_S}{N_H N_S-(N_H-1)(N_S-1)} \tag{2a,b}$$

where N_H and N_{NH} are the estimated MRIs of the peak wind effect r induced by hurricane and synoptic winds, respectively. Figure 2 shows examples of structural response under hurricane and synoptic winds: the peak inter-story drift in a building's principal direction, and the resultant peak accelerations, as functions of MRI. Because most standards (e.g., ASCE 49-12 (2012)) allow synoptic wind models to be used as approach flow for hurricane winds, the same aerodynamic pressure data can be used for both hurricane and synoptic winds. For a region exposed to multiple wind hazards whose flow characteristics are



(a) Inter-story drift ratio (y dir.) (b) Acceleration (resultant)

Figure 2. Mixed wind effects on inter-story drift and acceleration (dashed line for hurricanes; dash-dotted line for synoptic winds; solid line for mixed winds), (Yeo 2011).

different (e.g., those in hurricanes and tornadoes), aerodynamic pressure data appropriate for each wind hazard should be employed in the analyses. Their peak joint wind effects with specified MRIs can be estimated by combining the peak wind effects of the individual wind storms. This procedure is currently under development.

5. Member sizing

The design process starts by assigning to the structural members tentative sizes based on the designer's experience. The structure is then subjected to simplified wind loads, e.g., static loads specified in a standard. The members' DCIs are determined accordingly; they are typically found to be smaller or larger than unity. The member sizes are modified accordingly, and the structure with the modified members is redesigned using the DAD or the ESWL procedure. Iterations of the design are performed until the members' peak DCIs are less than, but sufficiently close to unity, subject to the applicable constructability and serviceability constraints. Iterative designs based on the DAD or ESWL procedures were found to have the potential for significant material savings (Simiu and Yeo 2019, p. 280). Current research is concerned with the use of optimization algorithms suitable for use in conjunction with the DAD_ESWL software. Candidate algorithms being investigated may be based on the following approaches.

The first approach, denoted by Opt(1), is briefly described below. Denote by t the lower bound of the DCI below which the serviceability constraints are not satisfied. The optimization problem is then stated as follows:

(Opt 1):
Maximize $t = \min[DCI_1(x),\ DCI_2(x),\ ...,\ DCI_n(x)]$
under the constraints:
$t \le DCI_i(x) \le 1.0,\ i = 1, 2, ..., n$ (n = *total number of members*)
$a(x) \le u$
$d_j(x) \le v,\ j = 1,\ 2,\ ...,\ m$ (m = *total number of floors*)

where a is the peak top floor acceleration and the d_j is the peak inter-story drift ratio at floor j. The vector x of member dimensions is the variable of the optimization problem, the aim of which is to maximize the lower bound t of DCIs. The upper bounds of u and v are specified constants.

Assume that an initial vector of member dimensions x^0 obtained by preliminary calculations is given such that the above constraints are satisfied with $x = x^0$ and $t = t_0$. By solving numerically the above optimization problem we obtain a vector x^* of optimal member dimensions. Opt(1) is likely to yield good results if $DCI_1(x^*), DCI_2(x^*),\ldots, DCI_n(x^*)$ are relatively close to one another.

A second approach being investigated, denoted by Opt(2), is formulated as follows:

(Opt 2):
Minimize $l = \sum_{i=1}^{n}[1 - DCI_i(x)]$
under the constraints:
$DCI_i(x) \le 1.0,\ i = 1, 2, ..., n$
$a(x) \le u$
$d_j(x) \le v,\ j = 1, 2, ..., m$

The objective is to minimize the sum of the distance between the vectors $\{DCI_1(x), DCI_2(x), ..., DCI_n(x)\}$ and unity. On account of its superior efficiency, the interior point method (Potra and Wright 2000) is tentatively being considered for use for the solution of the Opt(2) optimization problem.

6. Wind load factors and augmented mean recurrence intervals

Two methods for specifying design wind effects have been accepted in the United States. The first method specifies the MRI N of the peak wind effect p_{pk} (e.g., a demand-to-capacity index), and requires that the N-year peak wind effect $p_{pk}(N)$ be multiplied by a wind load factor (WLF) $\gamma_w(N)$ larger than unity, that is, $p_{des}(N) = \gamma_w(N)\, p_{pk}(N)$. The MRI N depends upon the risk category of the structure being designed. For most typical structures N = 50 years. The wind load factor $\gamma_w(N)$ depends upon the uncertainty in the estimation of $p_{pk}(N)$, commonly expressed as CoV $[p_{pk}(N)]$, where CoV denotes coefficient of variation, and on the value of the safety index β_w in the expression

$$\gamma_w(N) = 1 + \beta_w \mathrm{CoV}[p_{pk}(N)]. \tag{3}$$

By consensus, for typical structures $\beta_w \approx 2$ (Ellingwood et al. 1980). The uncertainty in $p_{pk}(N)$ depends upon its component uncertainties (i.e., the uncertainties in the wind climatological, micrometeorological, aerodynamic, dynamic, and directional factors that determine the response). These uncertainties may vary from case to case (for example, the uncertainties in the wind speeds depend upon the length of the data record, in addition to other factors), meaning that so does CoV$[p_{pk}(N)]$ and therefore the wind load factor, even if β_w is kept constant.

So far estimates of most of those uncertainties have been largely subjective. These estimates, as well as the estimates of the WLFs, can therefore be affected by significant errors. For example, preliminary calculations based on objective estimation methods described by Masters et al. (2010) showed that the coefficient of variation of the velocity pressure exposure coefficient K_z, as defined in the ASCE 7 Standard (ASCE 2005; 2010; 2016), can in some cases be larger than the subjective estimate proposed in Ellingwood et al. (1980). Such differences can result in the underestimation of the wind load factor, and therefore of the wind effect of interest.

The second method for specifying design wind effects uses a WLF = 1 in conjunction with an *augmented mean recurrence interval* (AMRI) of the design peak wind effect, denoted by N_a, such that

$$p_{pk}(N_a) = \gamma_w(N) p_{pk}(N) . \quad (4)$$

Therefore,

$$U(z_{std}, z_{0std}, N_a) = \sqrt{\gamma_w(N)}\, U(z_{std}, z_{0std}, N) . \quad (5)$$

where, typically, z_{std} = 10 m and $z_{0\ std}$ = 0.03 m. Assuming the validity of the Extreme Value Type I distribution of the extreme wind speeds, it can be shown that

$$N_a = exp\left\{ \frac{\sqrt{\gamma_w(N)} - 1}{0.78} \left[\frac{1}{\mathrm{CoV}(U_i)} - 0.45 \right] + \sqrt{\gamma_w(N)} \ln(N) \right\} \quad (6)$$

where U_i is the sample of measured or simulated wind speeds. The ASCE 7-10 and 7-16 Standards disregard the dependence of $\gamma_w(N)$ on CoV[$p_{pk}(N)$] and the dependence of N_a on both $\gamma_w(N)$ and CoV(U_i). In both standards, to the WLF γ_w (50 yrs) = 1.6 for Risk Category II structures specified in the ASCE 7-05 Standard there corresponds an augmented MRI $N_a^{ASCE} \approx$ 700 years. Assume now CoV(U_i) = 0.16 and $\gamma_w(N)$ = 1.6. From Eq. 6 it then follows N_a = 1010 years. This example illustrates the sensitivity of N_a to variations in CoV(U_i). Assume, in addition, γ_w(50 yrs) > 1.6, owing, for example, to larger values of CoV(K_z) and/or other component uncertainties.

The foregoing example showes that standard values of γ_w or of the augmented MRI N_a should be used for design purposes only if an analysis of the uncertainty in the structural response demonstrates that those values are acceptable.

7. Example: a 60-story reinforced concrete building

Figure 3 shows a 60-story reinforced concrete building with rigid diaphragm floors whose dimensions are 45.7 m in width, 30.5 m in depth, and 182.9 m in height, known as the CAARC building (Melbourne 1980). The structure has a moment-resisting system consisting of columns, beams, and slabs. The floor plan has 7 bays by 5 bays along the width and the depth, respectively. The building is assumed to be sited in suburban terrain exposure in Kansas City, Missouri. The orientation angle of the building is 270° clockwise from the north, which means the front façade of the building faces the north. The modal damping ratios are assumed to be 2% in all six modes considered in this design. Second-order effects (i.e., *P-Δ* and *P-δ* effects) were accounted for. The information required for building modeling and analysis was provided in the DAD_ESWL "Bldg. modeling" panel, as shown in Fig. 4.

Structural response to wind was determined following the dynamic analysis performed within the framework of the DAD procedure. Time series of wind loads acting at each floor in the x, y and θ axes were determined from aerodynamic pressure coefficients induced by wind velocities with wind directions in 10° increments (0°, 10°, …, 350°) (Venanzi 2005). The length scale of the building model was 1:500. The duration of the aerodynamic pressure

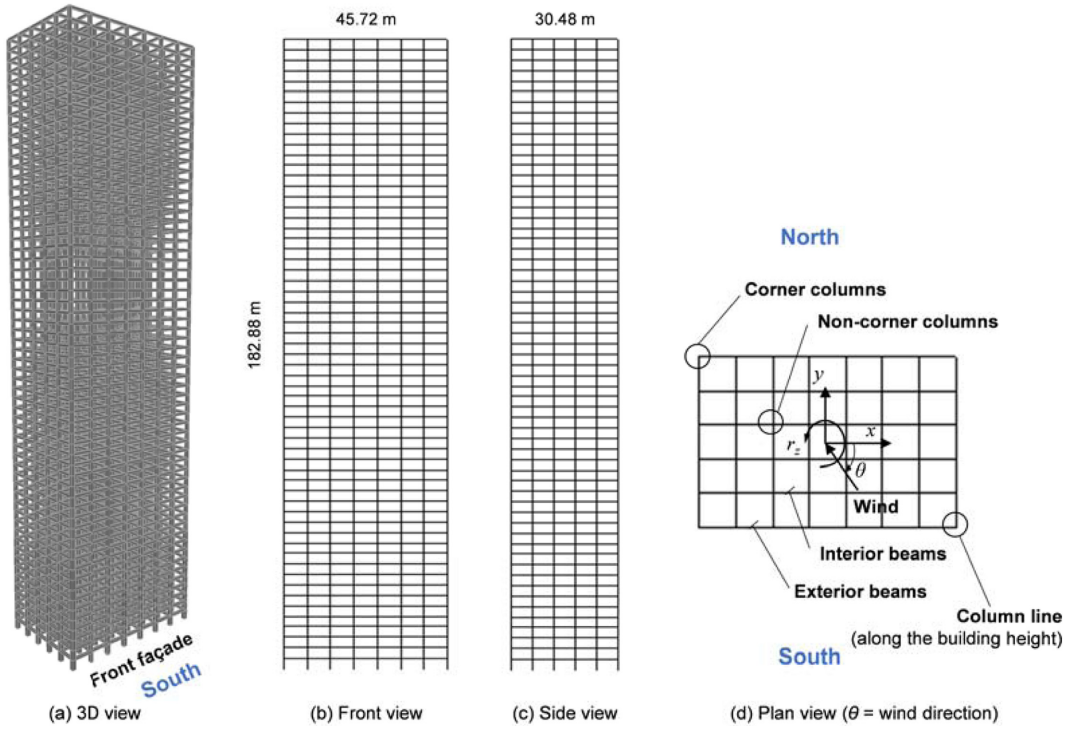


Figure 3. Schematic view of a reinforced concrete building.

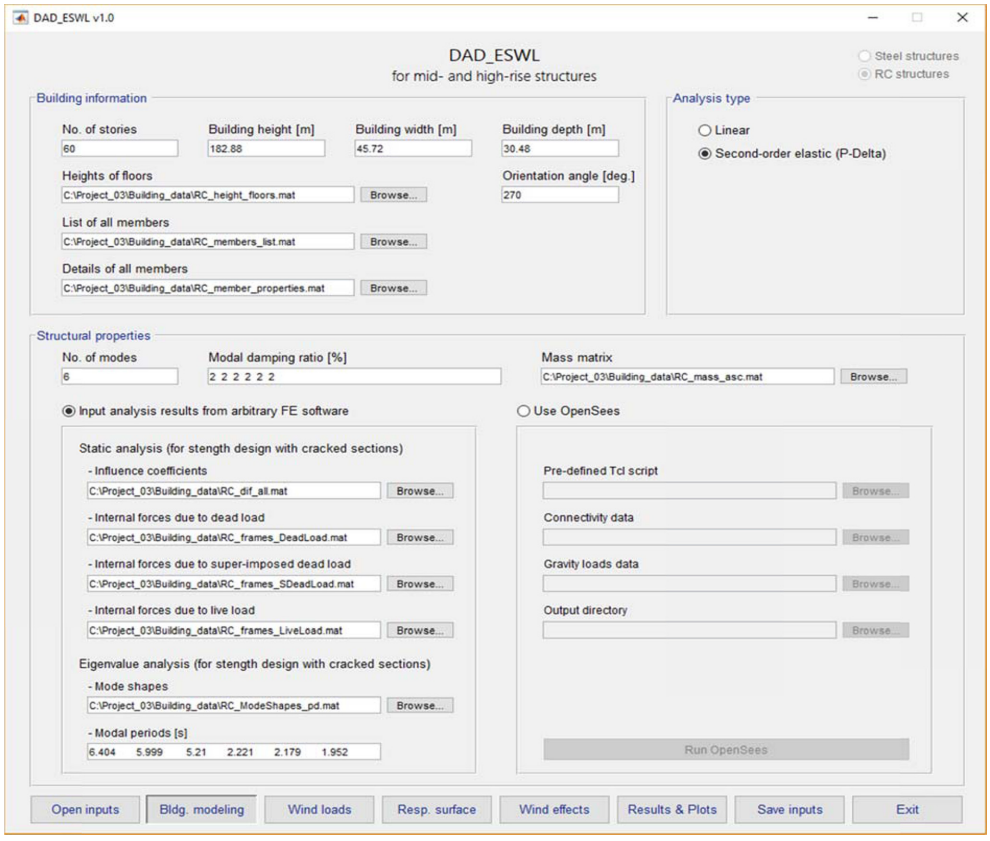


Figure 4. "Bldg. modeling" panel in DAD_ESWL software.

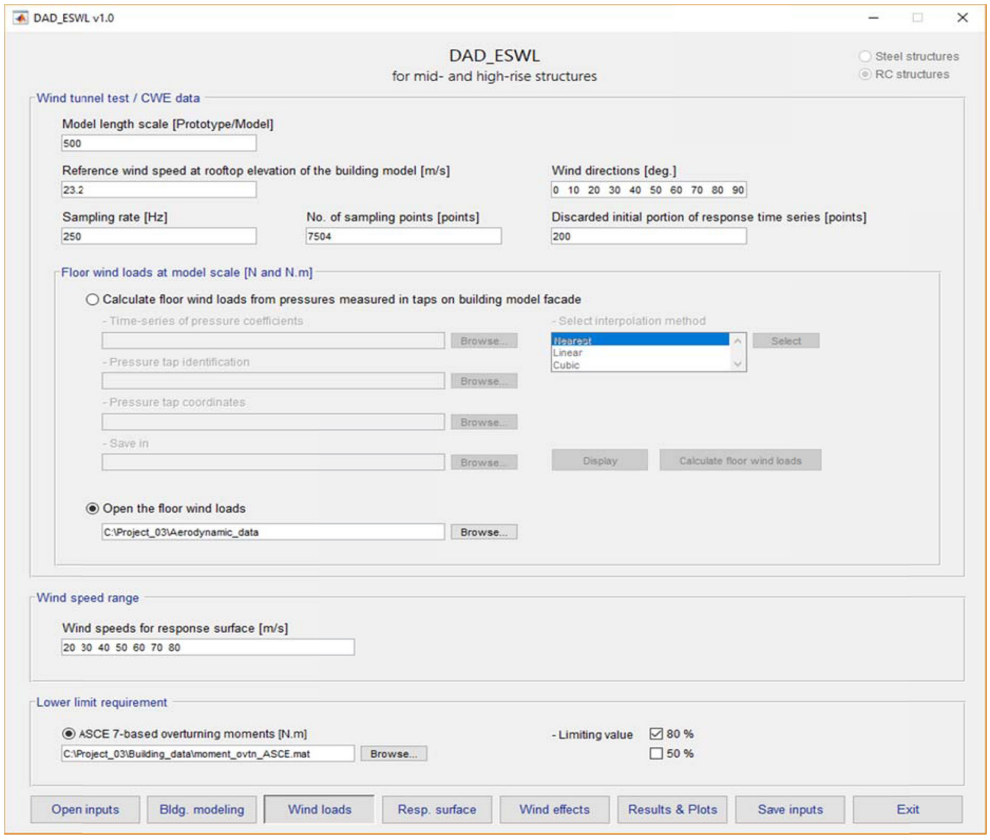


Figure 5. "Wind loads" panel in DAD_ESWL software.

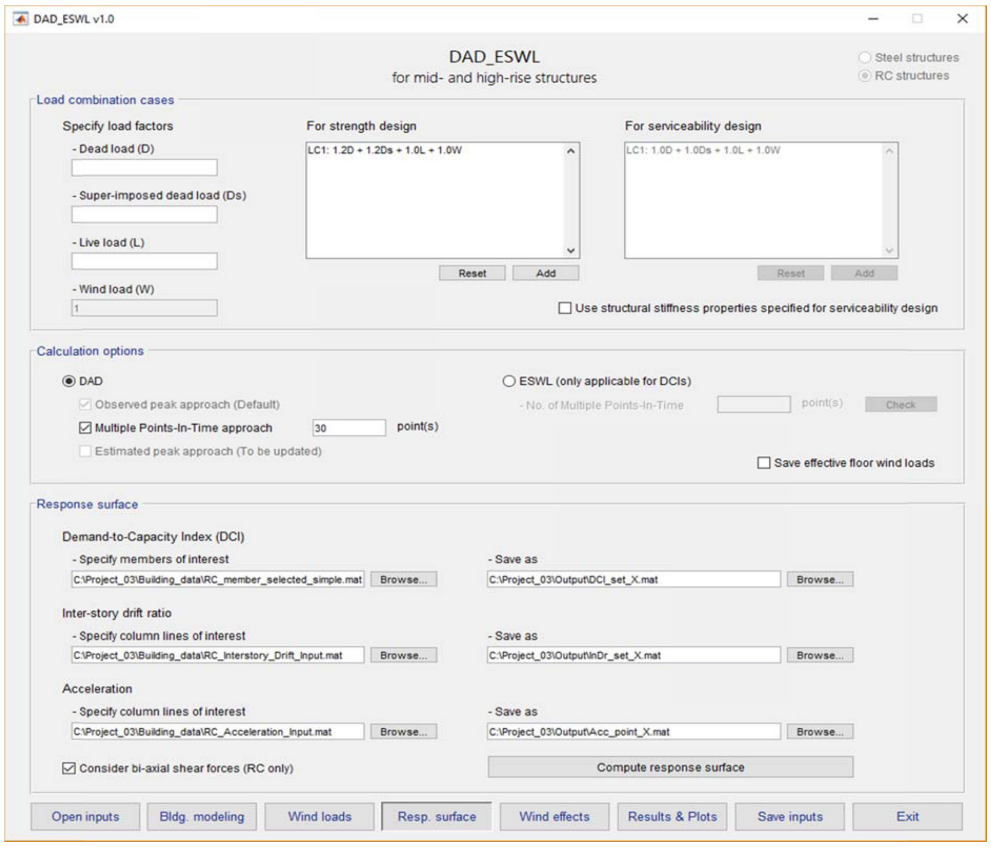


Figure 6. "Resp. surface" panel in DAD_ESWL software.

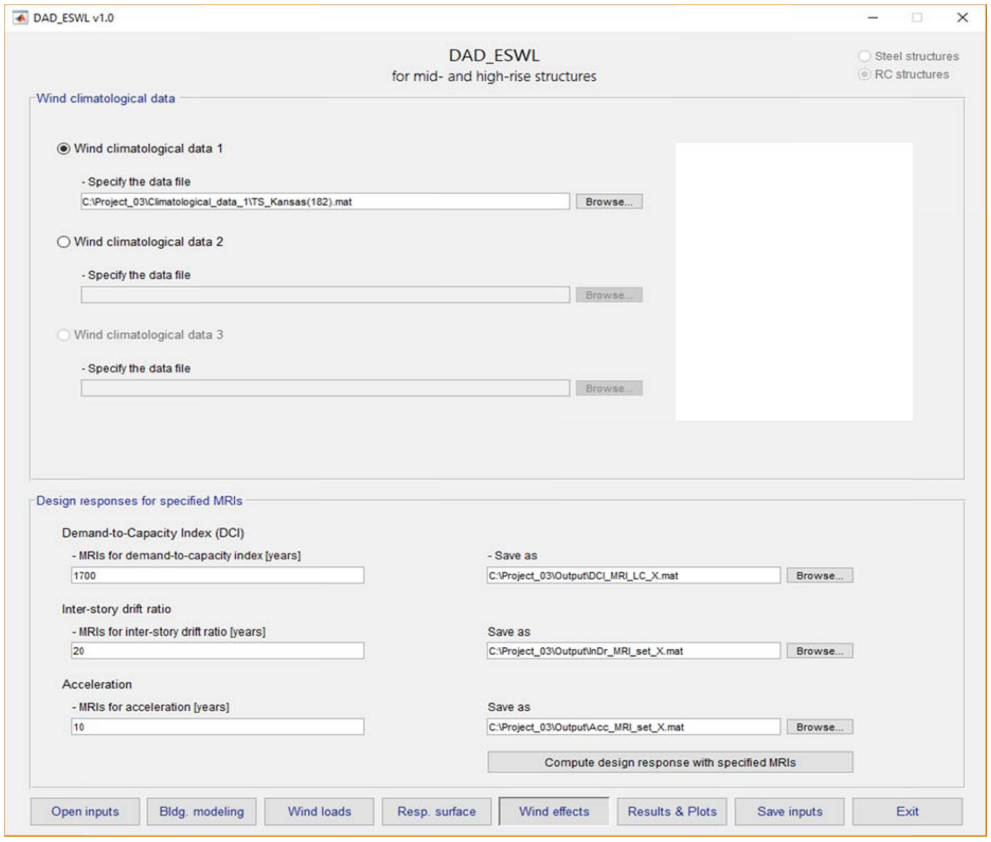


Figure 7. "Wind effects" panel in DAD_ESWL software.

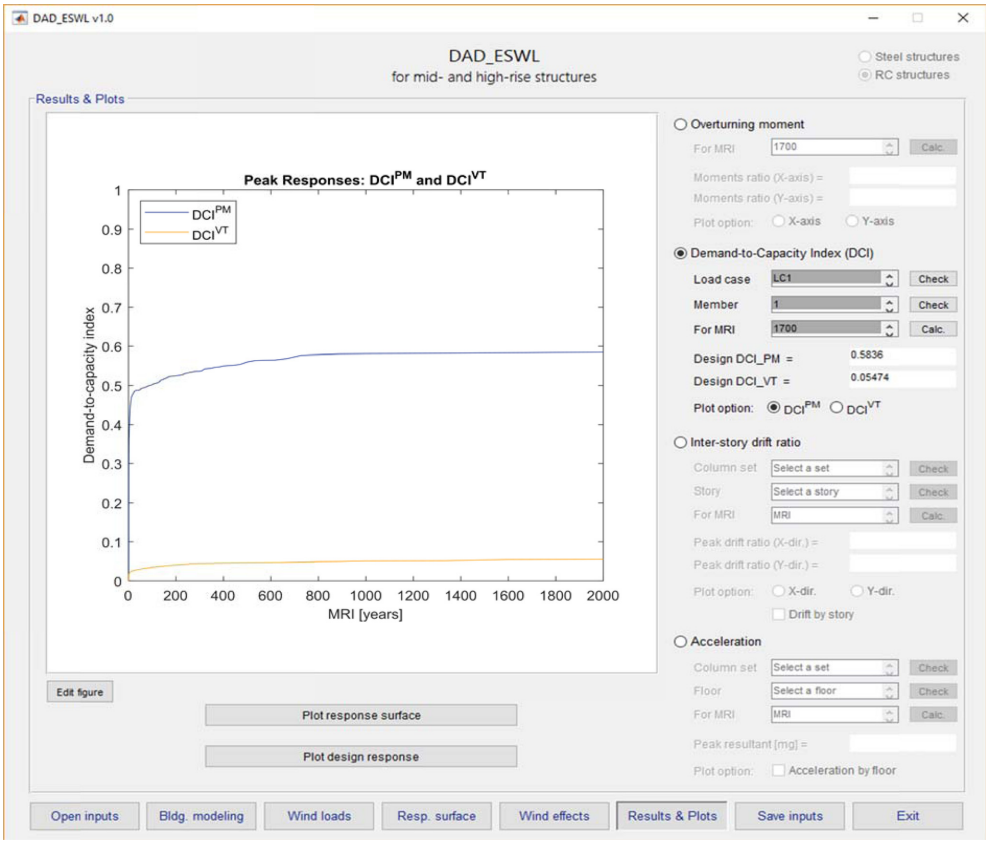


Figure 8. "Results & Plots" panel in DAD_ESWL software.

datasets was 30 s with sampling rate of 250 Hz, resulting in approximately 7500 samples for each pressure tap. The reference mean hourly wind speed was 23.2 m/s at top of the building model during the wind tunnel tests. Such information required for the analysis was inputted on the DAD_ESWL "Wind loads" panel, as shown in Fig. 5.

The building was analyzed for gravity and wind loads with the load combination $1.2D + 1.0L + 1.0W$ for strength design and $1.2D + 1.0L + 1.0W$ for serviceability design, where D is the total dead load, L is the live load, and W is the wind load. The multiple points-in-time approach with 30 points was used to determine peaks of resultant time series (Yeo 2013). Response surfaces of interest were calculated using the DAD option in the DAD_ESWL "Resp. surface" panel, as shown in Fig. 6.

The climatological wind dataset used in the computations was generated for Kansas City by Monte Carlo simulation using measured data (Yeo 2014). By combining the requisite response surfaces with the wind climatological dataset at the site, DCIs, inter-story drift ratios, and resultant acceleration were determined for 1700-, 20-, and 10-yr MRIs, respectively. Figure 7 shows the DAD_ESWL "Wind effects" panel for calculating the MRI-based design responses. Those results are visually summarized in the "Results & Plots" panel (Fig. 8).

8. Conclusions

The brief review of NIST research in the field of tall building design for wind presented in this work highlighted innovations achieved, following the development of pressure scanner technology, by taking advantage of current "big data" storage and processing capabilities. The innovations achieved so far include: the rational division of tasks between wind and structural engineers in the design process; the iterative multi-mode dynamic analyses required to account for successive changes in structural stiffness; the use of response surfaces and non-parametric statistics to account rigorously and transparently for wind directionality effects; and the automation of the process by which wind effects are determined given the wind climatological and aerodynamic data supplied by the wind engineering laboratory. Software by which the automation is achieved in practice is available in www.nist.gov/wind. Research in progress includes: the determination of design wind loads by accounting for estimates of uncertainty in those data; risk-consistent analysis for multiple wind hazards; and the integration in the design process of modern optimization algorithms. Results obtained so far confirm that values of the wind load factors and of augmented mean recurrence intervals specified in the ASCE 7-05 and the ASCE 7-16 Standard, respectively, should be used for design purposes

only if their adequacy is verified on the basis of information on uncertainty quantification supplied by the wind engineering laboratory.

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