EQUIVALENT STATIC WIND LOADS VS. DATABASE-ASSISTED DESIGN OF TALL BUILDINGS: AN ASSESSMENT

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Abstract:

Recent developments in pressure measurement technology, and unprecedented "big data" capabilities, have enabled the development of Database-assisted Design (DAD), a powerful innovative approach to the design of tall buildings for wind. DAD is accurate, rigorous, transparent, and user-friendly. Also, DAD eliminates unwieldy back-and-forth interactions between the wind and the structural engineer, needed in traditional practices if iterative designs are performed. In spite of these advantages, some structural engineers have shown interest in an alternative approach that uses equivalent static wind loads (ESWLs) in lieu of DAD. Such an approach is warranted if ESWLs induce in structural members demand-to-capacity indexes (DCIs) approximately equal to their peak counterparts obtained by DAD. This paper presents and assesses a simple procedure for calculating such ESWLs. The procedure uses an effective multiple points-in-time (MPIT) method for estimating combined peak wind effects, and accounts rigorously and transparently for wind directionality. A case study is presented that uses both the ESWL and DAD procedures, with the latter providing the requisite benchmark results. DCIs obtained from ESWLs based on the use of ten points-in-time (corresponding to 60 wind loading cases) were significantly closer to the benchmark DAD values than their counterparts based on the use of, e.g., four points-in-time (corresponding to 24 wind loading cases). For the building considered in this case study,

ESWL-based design DCIs approximated to within approximately 3 % the DCIs yielded by DAD. The approximation was found to be poorer for cases in which a single unfavorable wind direction is strongly dominant. The ESWL procedure is generally inapplicable to structures with complex shapes. In all cases, the DAD procedure is the safest and most risk-consistent design option.

Author keywords: Database-Assisted Design (DAD); demand-to-capacity indexes (DCIs); equivalent static wind loads (ESWLs); iterative design; tall buildings.

1. INTRODUCTION

The increasingly common use of multi-channel pressure scanners has led to the development of procedures for estimating aerodynamic wind loads on tall buildings based on data consisting of (i) simultaneously measured time series of pressure coefficients at large numbers of pressure taps on wind tunnel building models, and (ii) simulated sets of extreme directional wind speeds at the building site [1-4]. This paper describes the sequence of steps by which the structural design is performed using: (i) Database-Assisted Design (DAD), and (ii) a procedure, largely derived from DAD, wherein the design is based on Equivalent Static Wind Loads (ESWLs). The ESWL procedure is generally feasible only for structures with simple geometry (e.g., structures with a rectangular shape in plan). For structures with elaborate geometries (e.g., the CCTV building, the Shanghai World Financial Center, or the Burj Khalifa tower), the most appropriate design option is the use of the DAD approach.

In a rare case in which reports on the same buildings by different wind engineering laboratories ~~reports~~ were available for independent scrutiny, it was found that (i) estimates of wind effects on the same tall structure considered in the reports differed from each other by more than 40 %, and (ii) the reports lacked the transparency and traceability required to allow an understanding of the reasons for this major discrepancy [5]. The National Institute of Standards and Technology (NIST) was tasked with producing tools capable of yielding estimates of the requisite wind effects that could be readily and effectively scrutinized. The result of the efforts performed within the framework of this task was the development of DAD, a rigorous, transparent and effective procedure made possible by (i) the development of hardware capable of simultaneously measuring and recording time histories of pressures at multiple taps, and (ii) the availability of computer resources needed for processing large amounts of data economically and in a timely fashion [6, 7].

Static wind loads whose effects upon the structural system are reasonably close to those of the randomly fluctuating wind loads employed in DAD are called Equivalent Static Wind Loads. It is shown in this paper that DAD is as user-friendly as, and more accurate than its ESWL counterpart, although it is, within fully acceptable limits, more computer-intensive. This paper describes a procedure for estimating ESWL and notes its advantages and limitations. Unlike other procedures having the same objective, the ESWL procedure presented in this paper is assessed objectively against the benchmark values provided by DAD.

For both the DAD and the ESWL procedures, the wind climatological data and the simultaneous measurements of pressure coefficients at multiple taps, as well as estimates of the uncertainties in these data, are provided by the wind engineering laboratory. Once these data are delivered, the structural engineer is in full control of the design process. This allows the iterative design ~~process~~ to proceed smoothly and in timely fashion until the calculated demand-to-capacity indexes (DCIs) are acceptably close to unity. In particular, the availability of simultaneous pressure measurements at multiple taps renders obsolete the earlier practice, related to the use of the High Frequency Force Balance approach, that required parts of the dynamic analyses to be performed by the wind engineer. Both the DAD and EWSL procedures in this paper are user-friendly, transparent, readily subjected to effective public scrutiny, and easily integrated into Building Information Modeling (BIM) systems [8].

The DAD and the ESWL procedure are described in Section 2 and 3, respectively. Their respective steps are shown in the flowchart of Fig. 1. The estimation of ESWL is described in Section 4. Section 5 presents a case study. The Appendix describes the procedure for estimating wind effects with specified mean recurrence intervals by accounting for wind directionality.

2. DAD PROCEDURE

For the DAD procedure the structural engineer performs the following tasks:

1. Select the structural system, and determine its preliminary member sizes based on a simplified model of the wind loading (e.g., a static wind loading based on standard provisions). The structural design so achieved is denoted by D_0.

2. For the design D_0: determine the system's mechanical properties, including the modal shapes, natural frequencies of vibration, and damping ratios, as well as the requisite influence coefficients; and develop a lumped-mass model of the structure. P-Δ an P-δ effects can be accounted for by using, for example, the geometric stiffness matrix [9].

3. From the time histories of simultaneously measured pressure coefficients, determine the time histories of the randomly varying aerodynamic loads induced at all floor levels by mean wind speeds from, depending upon location, 10 m/s to 80 m/s in increments of 10 m/s, say, with directions from $0° \leq \theta < 360°$ typically in increments of 10°, say. The reference height for the mean wind speeds is typically assumed to be the height of the structure.

Tasks 4, 5 and 6 are performed for each of those directional wind speeds and are required for the determination, by accounting rigorously for directionality, of the requisite wind effects with the specified design mean recurrence interval (MRI), as shown subsequently.

4. Perform the dynamic analysis based on the lumped-mass model of the structure to obtain the time histories of the inertial forces induced by the respective aerodynamic loads, and the effective wind-induced loads consisting of the sums of the aerodynamic and inertial force time histories. The lateral loads are determined at all floor levels of the building.

5. For each cross section of interest, use the appropriate influence coefficients to obtain time series of the internal forces and the associated DCIs induced by the combination of effective floor wind loads determined in task 4 with factored gravity loads. The DCIs are the left-hand sides of the design interaction equations, and are typically used to size members subjected to more than one type of internal force. For example, the interaction equations for steel members subjected to flexure and axial forces are [10]:

$$\text{When } \frac{P_r}{\phi_p P_n} \geq 0.2, \qquad \frac{P_r}{\phi_p P_n} + \frac{8}{9}\left(\frac{M_{rx}}{\phi_m M_{nx}} + \frac{M_{ry}}{\phi_m M_{ny}}\right) \leq 1.0 \tag{1a}$$

$$\text{When } \frac{P_r}{\phi_p P_n} < 0.2, \qquad \frac{P_r}{2\phi_p P_n} + \left(\frac{M_{rx}}{\phi_m M_{nx}} + \frac{M_{ry}}{\phi_m M_{ny}}\right) \leq 1.0 \tag{1b}$$

In Eqs. 1a and 1b, P_r and P_n are the required and available tensile or compressive strength; M_{rx} and M_{nx} are the required and available flexural strength about the strong axis; M_{ry} and M_{ny} are the required and available flexural strength about the weak axis; ϕ_p and ϕ_m are resistance factors. The required strengths are based on combinations of wind and gravity effects specified in ASCE 7-16 [11] or other standards. A similar, though simpler expression for the DCI, is applied to shear forces and torsional moments.

6. Construct the response surfaces of the peak combined effects (e.g., DCIs, inter-story drift ratio, accelerations) as functions of wind speed and direction. For each of the directional wind speeds defined in task 3, determine for each cross section of interest the peak of the DCI time series (e.g., Eqs. 1a and 1b), and construct from the results so obtained a peak DCI response

surface. The response surface is a property of the aerodynamic and mechanical characteristics of the structure, independent of the wind climate, that provides for each cross section of interest the peak DCIs (or other wind effects) as functions of wind speed and direction.

7. Use the information contained in the response surfaces and in the matrices of directional wind speeds at the site to determine, by accounting for wind directionality, the design DCIs with the specified design MRI $\bar{N}$, denoted by $\mathrm{DCI}_m^{\mathrm{pk}}(\bar{N})$, for the member cross sections of interest *m*. The steps required for this purpose are described in detail in the Appendix.

In general, the preliminary design D_0 does not satisfy the strength and/or serviceability design criteria. The structural members are then re-sized to produce a modified structural design D_1. This iterative process continues until the final design is satisfactory.

Tasks 2 through 7 are repeated as necessary until the design DCIs are close to unity, to within serviceability constraints. Each iteration entails a re-sizing of the structural members consistent with the respective estimated design DCIs. Details are provided in Section 5.

3. EQUIVALENT STATIC WIND LOAD (ESWL) PROCEDURE

Like DAD, the ESWL procedure requires the wind engineer to provide (i) wind climatological data at the building site, (ii) time series of pressure coefficients measured simultaneously at multiple taps, and (iii) uncertainty estimates for both the wind climatological and the aerodynamic data. Once these three tasks are completed, the design process is fully the responsibility of the structural engineer.

The first four tasks of the ESWL procedure are identical to the first four tasks of the DAD procedure as listed in Section 1 and shown in Fig. 1. They are briefly summarized below.

1. Select the structural system, and determine its preliminary member sizes. The structural design so achieved is denoted by D_0.

2. Determine the system's mechanical properties for the design D_0, and develop a lumped-mass model of the structure.

3. From the time histories of simultaneously measured pressure coefficients, determine the time histories of the randomly varying aerodynamic loads induced at all floor levels by mean wind speeds U from direction θ, where 20 m/s $\leq U \leq$ 80 m/s in increments of 10 m/s, say, and $0° \leq \theta <$ 360° in increments of, say, 10°.

4. For each of the directional wind speeds defined in task 3, perform the dynamic analysis of the structure D_0 to obtain the time histories the effective wind-induced loads $F_{kq}(U,\theta,t)$ at floor k ($k = 1, 2, \ldots, n_f$; $q = x, y, \vartheta$) consisting of the sum of the inertial and aerodynamic forces applied at the floor lumped mass. The lateral loads determined in this task consist of the three components acting on the principal axes of x, y and ϑ (subscript q).

The subsequent tasks are performed for each of the wind speeds and directions defined in task 3.

4a. Determine the static loads $F_{kx,p}^{\mathrm{ESWL}}(U,\theta)$, $F_{ky,p}^{\mathrm{ESWL}}(U,\theta)$, and $F_{k\vartheta,p}^{\mathrm{ESWL}}(U,\theta)$ acting at the center of mass of floor k in the direction of the building's principal axes x, y and in torsion, respectively, where the subscript p ($p = 1, 2, \ldots, p_{\max}$) identifies distinct wind loading cases WLC$_p$ associated with superpositions of ESWL load effects, and $p_{\max}$ is a function of the number n_{pit} of points in time used to obtain the peak effects of interest. This task is described in detail in Section 4.

5. For each member cross section m of interest, calculate the internal forces used to determine its DCI, and substitute their expressions into the expressions for the DCIs (e.g., Eqs. 1). This task requires the use of the static wind loads determined in task 4a, the factored gravity loads, and the influence coefficients $r_{mk,x}$, $r_{mk,y}$, and $r_{mk,\vartheta}$. The influence coefficients represent internal forces at

cross section m induced by a unit floor load in the direction x, y, and ϑ applied at floor k, respectively. Denote the internal forces by $f_{m,p}^{\text{ESWL}}(U,\theta)$. Their expression is

$$f_{m,p}^{\text{ESWL}}(U,\theta) = \sum_{k=1}^{n} r_{mk,x} F_{kx,p}^{\text{ESWL}}(U,\theta) + \sum_{k=1}^{n} r_{mk,y} F_{ky,p}^{\text{ESWL}}(U,\theta) + \sum_{k=1}^{n} r_{mk,\vartheta} F_{k\vartheta,p}^{\text{ESWL}}(U,\theta) \quad (2)$$

The corresponding demand-to-capacity indexes, denoted by $\text{DCI}_{m,p}^{\text{ESWL}}(U,\theta)$, are obtained by substituting internal forces determined by Eq. 2 into the expressions for the DCIs. For design purposes only the largest of these DCIs is of interest, that is,

$$\text{DCI}_{m}^{\text{RS,ESWL}}(U,\theta) = \max{}_{p}(\text{DCI}_{m,p}^{\text{ESWL}}(U,\theta)) \quad (3)$$

6. Construct the response surfaces representing, for each cross section m of interest, the dependence of its $\text{DCI}_{m}^{\text{RS,ESWL}}(U,\theta)$ upon wind speed U and direction θ.

7. Use the response surfaces constructed in task 6 and the non-parametric statistical procedure described in detail in the Appendix to determine, from the values $\text{DCI}_{m}^{\text{RS,ESWL}}(U,\theta)$ and the climatological data $[U_{ij}]$ at a site of interest (for the detail of $[U_{ij}]$ see Appendix A1), the design DCIs with an $\bar{N}$-year mean recurrence interval, $\text{DCI}_{m}^{\text{pk}}(\bar{N})$. Depending upon the uncertainties in the wind velocity and aerodynamic data, as determined by the wind engineering laboratory, the design MRI may have to differ from the value specified in the ASCE 7-16 Standard [11], in which case it can be determined as in [12] or by a similar method.

If the design DCIs determined in task 7 differ significantly from unity, the structure's members are re-sized to create a new design D_1. Tasks 2 to 7 are then performed on that design. This process is iterated until a structural design is achieved for which, in each structural member, the design DCI is sufficiently close to unity, to within serviceability constraints.

In principle, DAD and ESWL can be applied to buildings undergoing aeroelastic effects under sufficiently strong winds provided that the aerodynamic pressures are measured on aeroelastic models [13]. Currently, DAD and ESWL are developed only for structures for which aeroelastic effects are negligible. Such structures typically include those for which (1) the lowest velocity capable of inducing aeroelastic effects is the critical velocity associated with vortices with frequency equal to the fundamental natural frequency of vibration of the structure, and (2) the largest velocity that can occur during the anticipated life of the structure is lower than that critical velocity [14].

4. ESTIMATION OF EQUIVALENT STATIC WIND LOADS

This section describes a simple approach to structural design, wherein the peak DCIs produced by randomly fluctuating effective wind forces are replaced by their counterparts produced by equivalent static wind loads. For ease of exposition, the indexes U and θ will be omitted in this section.

First, the structure is assumed to be subjected to wind loads acting only in one direction x at its centers of mass. For any given wind speed, let the effective (i.e., aerodynamic plus inertia) randomly fluctuating load at floor k acting along the principal axis x of the building be denoted by $F_{kx}(t)$, where k = 1, 2, …, n_f (Fig. 2). The corresponding overturning moment at the base of the building is (Fig. 2a)

$$M_{by}(t) = h[F_{1x}(t) + 2F_{2x}(t) + \cdots + nF_{nx}(t)] \tag{4}$$

where it is assumed for simplicity that all floors have the same height h. The peak of $M_{by}(t)$, denoted by $\max_t[M_{by}(t)]$, occurs at time t_{1x} (Fig. 2a). The equivalent static wind load in direction x at floor k, denoted by F_{kx}^{ESWL} is therefore

$$F_{kx}^{\mathrm{ESWL}} = F_{kx}(t_{1x}) \tag{5}$$

and the moment at the structure's base is

$$h(F_{1x}^{\mathrm{ESWL}} + 2F_{2x}^{\mathrm{ESWL}} + \cdots + nF_{nx}^{\mathrm{ESWL}}) = M_{by}(t_{1x}) \tag{6}$$

The static wind loading determined as described above may be called equivalent static wind loading if it induces in all structural members DCIs approximately equal to their peak DAD counterparts.

The internal force at the member cross section m induced by the effective fluctuating forces $F_{kx}(t)$ at floor k ($k = 1, 2, \ldots, n_f$) can be written as

$$f_m(t) = \sum_{k=1}^{n} r_{mk,x} F_{kx}(t) \tag{7}$$

where $r_{mk,x}$ is the influence coefficients representing the internal forces at cross section m induced by a unit load applied at floor k. The ESWLs F_{kx}^{ESWL} are determined as in Eq. 5. The fundamental assumption of this approach is that the peak internal forces occur at the same time t_{1x} as the peak base moment. If this were the case, the following system of equations would be satisfied:

$$\max_t [\, f_m(t)] = \sum_{k=1}^{n} r_{mk,x} F_{kx}^{\mathrm{ESWL}} \qquad (m = 1, 2, \ldots, m_{max}) \tag{8}$$

Equation 8 would be rigorously true if the influence coefficients $r_{mk,x}$ were proportional to the floor height kh, which is not the case. However, it is shown in Section 5 that Eq. 8 (and, in particular, its counterpart wherein forces in the principal direction y and torsional moments about the center of mass are also acting on the structure) can be satisfied to within a close approximation if wind directionality effects are accounted for. Note that this may not be the case if wind loads are dominant in one direction only, or for complex structural system, as noted in the Introduction.

Since structures experience wind-induced forces simultaneously in the directions x and y of the structure's principal axes, as well as wind-induced torsional moments about the center of mass, wind effects are due to the superposition of the effects of these three actions. It is therefore possible to construct, from the time histories of those three individual wind effects, the time history of their combined effect via simple summation, and estimate the peak of that time history. This approach can be used in the DAD procedure.

For the ESWL, a multiple points-in time (MPIT) [15] approach with a number n_{pit} of points in a time series is used, as follows (see Fig. 3). The highest peak of the time series of the base moments induced by forces in the x direction, and its time of occurrence t_{1x}, are identified. The forces $F_{kx}(t_{1x})$ are defined as the *principal* ESWLs at floors k (k = 1, 2, ..., n_f) acting on the structure at time t_{1x}. The forces $F_{ky}(t_{1x})$ and $F_{k\vartheta}(t_{1x})$ are defined as the *companion* ESWLs at floors k. Because the simultaneously acting forces $F_{kx}(t_{1x})$, $F_{ky}(t_{1x})$ and $F_{k\vartheta}(t_{1x})$ do not necessarily produce the most unfavorable effect being considered, it is necessary to perform the operation just described for the second, …, n_{pit}-th highest peak of the time series of the base moments induced by forces in the x direction. Thus, a total of n_{pit} combinations of principal ESWLs and two companion ESWLs are obtained for times t_{1x}, t_{2x}, …, $t_{n_{pit}x}$.

Next, n_{pit} combinations of principal and companion ESWLs acting at times t_{1y}, t_{2y}, …, $t_{n_{pit}y}$ are obtained by considering the highest, second highest, …, n_{pit}-th highest peak of the base moments induced by the forces in the direction y. A third set of n_{pit} combinations acting at times $t_{1\vartheta}$, $t_{2\vartheta}$, …, $t_{n_{pit}\vartheta}$ corresponds to the highest peaks of base moments induced by torsion. Three additional sets correspond to lowest peak base moments. Therefore, $p_{max} = 6n_{pit}$ sets of equivalent static wind loading combinations must be used to determine the wind effects of interest. The design wind effect is the largest of the effects induced by these $6n_{pit}$ sets of ESWLs.

The operations listed in this section are performed automatically using the ESWL option of the DAD_ESWL version 1.0 (www.nist.gov/wind).

5. CASE STUDY: APPLICATION TO DESIGN OF A 47-STORY STEEL BUILDING

This section presents as a case study the iterative design of a steel building wherein both the ESWL approach and the DAD approach are used to calculate member DCIs. The results of the ESWL procedure and the benchmark results provided by the DAD procedure are then compared to verify the acceptability of the ESWL calculations, and a discussion is presented on the relative advantages of ESWL and DAD.

5.1 Description of the structure

The structure being considered is a 47-story steel building with a square shape in plan and 40 m × 40 m × 160 m in depth, width and height, respectively. The structure has rigid diaphragm floors, outriggers and belt trusses (Fig. 4). The building's supports are assumed to be fixed. The structure consists of 2303 columns, 3948 beams, and 1152 diagonal braces. Columns are divided into three types: core, external core, and interior columns. Beams are divided into three types: exterior, internal, and core beams. Diagonal bracings are divided into two types: core and outrigger bracings. Each type of structural member has the same dimensions for 10 successive floors of the building's lowest 40 floors, and for the 7 highest floors. The columns and bracings consist of built-up hollow structural sections (HSS), and the beams consist of rolled W-sections selected from the AISC Steel Construction Manual [16]. The structure was assumed to be sited in open terrain exposure near the shore line at milepost 1950 in South Carolina (for a map showing milepost locations see www.nist.gov/wind). The orientation angle of the building is 270° clockwise from the north, that is, a façade of the building faces east. The aerodynamic pressure time histories were obtained from the Tokyo Polytechnic University (TPU) high-rise building aerodynamic database (http://www.wind.arch.t-kougei.ac.jp/system/eng/contents/code/tpu). Wind direction is defined by the clockwise angle θ, with the positive x-axis heading east, and the y-axis heading north (Fig. 4d).

The DCIs considered subsequently are induced by combinations of axial forces and bending moments.

5.2 Database-assisted design

A *preliminary design* of the structure, denoted by D_0, was performed, based on a simplified wind loading model for buildings of all heights experiencing dynamic along-wind response [11]. Second-order effects (P-Δ — member chord rotation effect and P-δ — member curvature effect) were accounted for by using geometric stiffness approach [9]. The natural frequencies of vibration for design D_0 of the structure are listed in Table 1. The modal damping ratios were assumed to be 1.5 % in all six modes.

For wind speeds of 20 m/s to 80 m/s in increments of 10 m/s and wind directions of 0° to 350° in increments of 10°, measured pressure coefficient time histories were used to determine time histories of applied aerodynamics loads at each floor. This step is necessary for the rigorous and transparent estimation of the wind effects with the specified MRI (e.g., design DCIs) by accounting for (i) wind directionality effects and (ii) the properties of the structure inherent in its final, rather than the preliminary, design.

Next, for each of the wind speeds and directions considered above, dynamic analyses were performed to obtain the respective time histories of the inertial forces induced by the time-varying aerodynamic loads. The effective wind-induced loads acting on the structure consist of the sums of the aerodynamic and inertial force time histories. It was assumed that the specified mean recurrence interval (MRI) of the demand-to-capacity indexes for the structure is 1700 years.

The peak DCIs were obtained from the DCI time histories induced by the effective wind loads combined with gravity loads as specified in the ASCE 7-16 Standard [11]:

$$1.2D + 1.0L + 1.0W \tag{9}$$

where D, L and W denote dead load, live load, and wind load, respectively. The interaction equations of DCI for steel members were taken from AISC 360-10 [10]. This step results in the construction of response surfaces for the peak DCIs of the members of interest for the specified range of wind speeds (20 m/s to 80 m/s) and wind directions (0° to 350°).

As the last step pertaining to the design D_0, the peak DCIs based on the load combination (Eq. 9) were used to estimate the design DCIs with a 1700-year MRI, as shown in the Appendix.

5.3 ESWL-based design

The first four steps of the ESWL-based design procedure are identical to the first four DAD steps, as listed in Sections 2 and 3 and shown in Fig. 1. The next step in the ESWL-based approach consists of determining the ESWLs for each of the multiple points in time (MPIT) wind loading cases (WLCs) for each wind speed and direction considered in task 4a (see Section 2). The wind loading cases are discussed in detail in Section 3. There follows the calculation, for each wind speed and direction, of the DCIs induced by the gravity loads in combination with each of the 60 (i.e., 6 × 10) wind loading cases (WLCs) that correspond to the selected number of points in time $n_{pit} = 10$. For each wind speed and direction, the largest of the DCIs induced by the 60 wind loading cases is selected and used to construct the DCI response surface. Figure 5 shows three components of the equivalent static floor wind loads, i.e., one principal and two companion ESWLs, based on the highest peak of each effective overturning moment and torsion at base.

The last step pertaining to the design D_0 is the use of the approach presented in the Appendix for determining the design DCIs with a 1700-year mean recurrence interval.

5.4 Results

5.4.1 Design iterations

The design was iterated to achieve design DCI values with a 1700-year MRI sufficiently close to unity. Table 2 lists the design DCIs calculated for designs D_0, D_1, and D_2 by both the DAD and the ESWL procedures. It is seen that the preliminary design D_0 resulted in DCIs significantly different from unity (e.g., for the ESWL procedure, DCI = 0.29 for the internal column (CI) at the 45th floor and 2.01 for the core column (COL) at the 1st floor).Through resizing of structural members, the DCI values reached levels judged to be satisfactory at the final design D_2. The natural frequencies of vibration increased by up to approximately 13 % from design D_0 to design D_2 (see Table 1). The ESWLs acting at each floor in the three principal directions changed from design D_0 to design D_2 as shown in Fig. 6. As explained in Section 4, peak base overturning and torsional moments determined by ESWL on the one hand and by DAD on the other are identical. The peak base shears determined by ESWL were found to be smaller by approximately 1 % than their DAD counterparts.

Table 1. Dynamic properties for designs D_0 through D_2

Mode	Natural frequencies of vibration [Hz]		
	D_0	D_1	D_2
1 (*y*-dir.)	0.218	0.250	0.246
2 (*x*-dir.)	0.218	0.250	0.246
3 (ϑ-dir.)	0.223	0.258	0.253
4 (ϑ-dir.)	0.568	0.664	0.638
5 (*x*-dir.)	0.644	0.724	0.708
6 (*y*-dir.)	0.644	0.724	0.708

5.4.2 Comparisons of DCIs based on DAD and ESWL procedures

Relative errors in the estimation of the DCIs by the ESWL procedure, defined as (DCI^{ESWL} – DCI^{DAD})/DCI^{DAD} × 100 (%), were calculated as functions of wind direction and wind speed for 60 selected structural members (24 columns, 28 beams, and 8 bracings) at four floor levels (15 members per floor; see Fig. 4d). Figure 7 shows the DCI response surface of the core column (COL) on the 45th floor under load combination (Eq. 9), colored by the relative errors in the estimation of the DCIs by the ESWL procedure. Maximum relative errors for the 24 columns, the 28 beams and the 8 bracings are plotted in Figs. 8a, 8b and 8c, respectively. As shown in Fig. 8, the largest relative errors of DCIs depend on the wind speed and wind direction and can be as high as -20 % in some cases. (For a few columns and bracings, the ratio of required to available axial load capacity was slightly larger than 0.2 if calculated by DAD and slightly lower than 0.2 if calculated by ESWL (see Eqs. 1). For the comparison between the respective DCIs to be meaningful, the DCIs were calculated in these cases by assuming that Eq. 1a was applicable for both the DAD and ESWL computations).

However, these differences are significantly reduced when wind directionality is considered in determining design DCIs with specified MRIs. As shown in Fig. 9, the design DCIs with MRI = 1700 years at milepost 1950 (South Carolina) obtained by the ESWLs were smaller by 3 % or less than their DAD counterparts.

Table 2. Design DCIs with 1700-year MRI based on DAD and on ESWL for designs D_0, D_1 and D_2.

Member ID*	Method	D_0				D_1				D_2			
		1st	17th	33rd	45th	1st	17th	33rd	45th	1st	17th	33rd	45th
CC	DAD	0.89	1.14	1.51	0.42	0.78	0.76	0.59	0.13	0.83	0.88	0.84	0.87
	ESWL	0.89	1.13	1.51	0.42	0.76	0.74	0.57	0.13	0.83	0.88	0.84	0.86
CEW	DAD	0.62	0.97	1.53	0.37	0.65	0.76	0.68	0.15	0.96	0.80	0.88	0.87
	ESWL	0.62	0.96	1.52	0.37	0.66	0.74	0.66	0.15	0.95	0.80	0.88	0.87
CI	DAD	0.81	1.08	1.64	0.30	0.84	0.85	0.77	0.13	0.83	0.90	1.00	0.80
	ESWL	0.81	1.07	1.62	0.29	0.84	0.84	0.75	0.13	0.83	0.90	0.99	0.79
COL	DAD	1.99	1.62	1.02	0.36	0.79	0.62	0.52	0.15	0.84	0.62	0.90	0.91
	ESWL	2.01	1.61	1.02	0.36	0.76	0.59	0.49	0.10	0.84	0.62	0.89	0.90
CES	DAD	0.78	1.18	1.73	0.39	0.67	0.78	0.69	0.15	0.72	0.90	0.95	0.89
	ESWL	0.78	1.17	1.69	0.38	0.67	0.75	0.66	0.14	0.72	0.90	0.95	0.88
COR	DAD	1.37	1.44	1.57	0.61	1.10	0.92	0.63	0.23	0.56	0.67	0.87	0.78
	ESWL	1.38	1.42	1.55	0.60	1.11	0.88	0.59	0.22	0.56	0.67	0.86	0.78
BESW	DAD	0.66	1.00	1.00	0.66	0.63	0.96	1.00	0.66	0.91	0.98	0.69	0.97
	ESWL	0.64	0.97	0.97	0.64	0.61	0.95	0.97	0.65	0.90	0.98	0.69	0.96
BES	DAD	0.65	0.96	0.93	0.60	0.62	0.93	0.92	0.57	0.62	0.95	0.96	0.83
	ESWL	0.63	0.93	0.91	0.58	0.60	0.92	0.90	0.56	0.61	0.94	0.96	0.83
BI	DAD	0.86	1.40	1.62	1.50	0.85	1.34	1.56	1.42	0.85	0.88	0.95	0.97
	ESWL	0.85	1.39	1.62	1.48	0.85	1.34	1.54	1.40	0.85	0.88	0.93	0.97
BOS	DAD	0.77	0.78	0.81	0.62	0.77	0.77	0.80	0.62	0.77	0.78	0.81	0.90
	ESWL	0.76	0.77	0.79	0.62	0.77	0.77	0.79	0.61	0.77	0.77	0.80	0.90
BEWS	DAD	0.70	1.18	1.26	0.88	0.56	0.95	1.05	0.74	0.87	0.68	0.77	0.75
	ESWL	0.67	1.13	1.24	0.87	0.58	0.95	1.00	0.71	0.85	0.67	0.77	0.75
BEW	DAD	0.69	1.17	1.26	0.79	0.56	0.96	1.02	0.67	0.87	0.69	0.74	0.99
	ESWL	0.67	1.13	1.19	0.77	0.57	0.96	1.01	0.67	0.85	0.68	0.74	0.99
BOW	DAD	0.82	0.87	0.95	0.65	0.76	0.80	0.87	0.63	0.78	0.82	0.89	0.93
	ESWL	0.83	0.86	0.93	0.64	0.77	0.80	0.86	0.63	0.78	0.82	0.89	0.92
XOS	DAD	0.73	0.76	0.79	0.35	0.67	0.70	0.79	0.35	0.69	0.71	0.85	0.80
	ESWL	0.74	0.70	0.73	0.33	0.70	0.64	0.67	0.31	0.69	0.71	0.83	0.80
XOE	DAD	0.82	0.71	1.10	0.44	0.72	0.63	0.86	0.35	0.68	0.62	0.46	0.82
	ESWL	0.83	0.71	1.03	0.42	0.73	0.60	0.80	0.35	0.68	0.60	0.45	0.82

* CC = corner column; CEW = external column at west side of the building plan; CI = internal column; COL = core column at left side of the core; CES = external column at south; COR = core column at right side of the core; BESW = external beam at southern west; BES = external beam at south; BI = internal beam; BOS = core beam at south; BEWS = external beam at western south; BEW = external beam at west; BOW = core beam at west; XOS = core bracing at south; XOE = core bracing at east. See Fig. 4d for details.

The errors in the ESWL estimates of the DCI were calculated for various values of n_{pit}. As shown in Fig. 10, the estimation errors are negligible for $n_{pit} \geq 10$ are used in this case study. The ESWL procedure cannot overestimate the peak wind effects. Figure 10 is consistent with this fact.

To assess the efficiency of the ESWL procedure, the ratio r between ESWL and DAD computational times required to calculate design DCIs with MRI = 1700 years was obtained as functions of (i) the number of points n_{pit}, and (ii) the number of members being analyzed. The dependence of the ratio r upon n_{pit} was found to be almost negligible. For 60 members r was approximately 0.4. The relative efficiency of the ESWL procedure increases when larger numbers of structural members are considered. For 1000 members r was approximately 0.2. However, computation times for the DAD calculations were found to be fully compatible with practical capabilities of structural design offices.

6. SUMMARY AND CONCLUSIONS

This paper presents and assesses a simple procedure for calculating ESWLs that uses a multiple points-in-time (MPIT) approach for the estimation of peak wind effects, and accounts rigorously and transparently for wind directionality. As is the case for DAD, the calculation of the ESWLs is automated, user-friendly, transparent, and easily integrated into Building Information Modeling (BIM) systems. Case studies were performed by using both the ESWL and DAD procedures, with the latter providing the requisite benchmark results. The following conclusions were drawn from this work:

1) The use of 10 points in the multiple points-in-time (MPIT) estimator used in the procedure for calculating ESWLs increases the accuracy of the results with respect to the case where one or four points-in-time estimates (corresponding to 6 and 24 wind loading cases, respectively) are used. For the building considered in this study, the use of n_{pit} = 10 points approximates the design

demand-to-capacity indexes (DCIs) of members to within approximately 3 %. The differences could be larger if a single unfavorable wind direction is strongly dominant or if the structural system is complex.

2) The ESWL procedure allows the structural engineer to be in full control of the structural design. In particular, the availability of simultaneous pressure measurements at multiple taps renders obsolete the earlier practice, related to the use of the High Frequency Force Balance approach, that required parts of the dynamic analyses to be performed by the wind engineer. This division of tasks has two advantages. First, the capability to perform thorough dynamic analyses available to structural design offices is typically stronger than its wind engineering laboratory counterpart, resulting in more accurate dynamic analyses. Second, once the requisite wind climatological and aerodynamic data, as well as estimates of their respective uncertainties, are provided by the wind engineer, the structural engineer can perform the iterative design process smoothly, with no unnecessary and time-consuming back-and-forth with the wind engineer until the calculated DCIs are sufficiently close to unity.

3) The DAD procedure provides benchmarks against which the accuracy of ESWL calculations can be assessed and is a practical approach to the design of structures with complex structural systems for which the ESWL approach is not feasible.

4) It has been argued that some structural engineers may prefer performing the design for wind by using ESWLs, even though DAD is more accurate. Since both the DAD and the ESWL procedures are automated, the amount of labor required on the part of the structural engineer is the same, regardless of which procedure is used. In addition, it is worth noting that while design for seismic loads was originally based on static seismic loads, structural engineering culture has

evolved to the point where this is no longer necessarily the case. Design for wind may be expected to undergo a similar evolution.

As noted in the paper, the ESWL procedure should only be applied to structures with a simple shape in plan. Also, the quality of its performance, which was found to be fully satisfactory in the case study discussed in the paper, may depend on the type of structural system being considered and on the directional characteristics of the wind climate.

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APPENDIX. ESTIMATION OF WIND EFFECTS WITH SPECIFIED MEAN RECURRENCE INTERVALS [7]

A1. Matrix of largest directional wind speeds and mean annual rate of storm arrival

The notation $R_m(U_{ij})$ identifies the wind effect being considered (e.g., base moment, base shear, internal force, DCI, displacement, acceleration), and the indexes i and j identify the storm event and the wind direction, respectively. The peak wind effect in storm i is denoted by $\{R_{m,i}^{\mathrm{pk}}\}$. The peak wind effect with an $\bar{N}$ -year mean recurrence interval, denoted by $R_m^{\mathrm{pk}}(\bar{N})$, is determined by using information available in (i) the matrix of directional extreme wind speeds [U_{ij}], and (ii) the mean rate of arrival of storms per year, denoted by λ. Consider, for illustrative purposes, the 3 by 4 matrix of wind speeds (in m/s) at the site of the structure.

$$[U_{ij}] = \begin{bmatrix} 34 & \mathbf{45} & 32 & 44 \\ 37 & 39 & 36 & \mathbf{51} \\ 42 & 44 & 35 & \mathbf{46} \end{bmatrix} \tag{A1}$$

Under the convention inherent in the notation [U_{ij}] this matrix corresponds to three storm events and four wind directions, that is, i = 1, 2, 3 and j = 1, 2, 3, 4. For example, the wind speed that occurs in the second storm event from the third direction is U_{23} = 36 m/s. (The entries in the wind speed matrix could, for example, be mean hourly speeds at the elevation of the top of the structure with direction j over terrain with suburban exposure.) In the matrix of Eq. A1 the largest wind speeds in each of the three storms are indicated in bold type.

***A2. Transformation of matrix* [U_{ij}] *into matrix of wind effects* [$R_m(U_{ij})$]**

Transform the matrix $[U_{ij}]$ into the matrix by substituting in it the quantities $R_m(U_{ij})$ for the quantities U_{ij}. Assume that the result of this operation is the matrix

$$[R_m(U_{ij})] = \begin{bmatrix} 0.70 & \mathbf{1.02} & 0.80 & 0.68 \\ 0.83 & 0.77 & \mathbf{1.01} & 0.91 \\ \mathbf{1.07} & 0.98 & 0.96 & 0.74 \end{bmatrix} \tag{A2}$$

A3. Transformation of matrix of wind effects* [$R_m(U_{ij})$] *into vector

The peak wind effects induced by the wind speeds occurring in storm i depend upon the wind direction j. It is only the largest of those wind effects, that is, $\max_j[R_m(U_{ij})]$ (i = 1, 2, 3), that are of interest from a design viewpoint. These largest wind effects, shown in bold type in Eq. A2, form a vector $[1.02, 1.01, 1.07]^T$, where T denotes transpose. Note that $\max_j[R_m(U_{ij})]$ is not necessarily induced by the speed $\max_j(U_{ij})$. For example, $\max_j[R_m(U_{3j})]$ = 1.07 is not induced by the speed $\max_j(U_{3j})$ = U_{34} = 46 m/s, but rather by the speed U_{31} = 42 m/s.

The vector components $\max_j(R_m(U_{ij}))$ constitute the sample of the largest peak wind effects occurring in each of the i storm events (in this example i = 1, 2, 3), denoted by $\{R_{m,i}^{\text{pk}}\}$. The estimation of the response with any specified MRI ($R_m^{\mathrm{pk}}(\bar{N})$) is based on this sample, used in conjunction with the mean annual rate of occurrence of the storms.

$\mathrm{DCI}_m(U,\theta,t)$ = Time series of demand-to-capacity index (DCI) at cross section m, induced by the effective wind-induced forces $F_{kq}(U,\theta,t)$ ($q = x, y$, and ϑ) at floors k (k = 1, 2, …, n_f) for given values of U and θ

$\mathrm{DCI}_{m,p}^{\mathrm{ESWL}}(U,\theta)$ = ESWL-based DCI at cross section m, for wind loading case p (WLC$_p$) for given values of U and θ

$\mathrm{DCI}_m^{\mathrm{RS}}(U,\theta)$ = **Response Surface** representing the peak of the time series $\mathrm{DCI}_m(U,\theta,t)$, as a function of U and θ:
$\mathrm{DCI}_m^{\mathrm{RS}}(U,\theta) = \max_t(\mathrm{DCI}_m(U,\theta,t))$

$\mathrm{DCI}_m^{\mathrm{RS,ESWL}}(U,\theta)$ = **Response Surface** representing the maximum of $\mathrm{DCI}_{m,p}^{\mathrm{ESWL}}(U,\theta)$ for $p_{\max}$ wind loading cases, as a function of U and θ:
$\mathrm{DCI}_m^{\mathrm{RS,ESWL}}(U,\theta) = \max_p(\mathrm{DCI}_{m,p}^{\mathrm{ESWL}}(U,\theta))$

$F_{kq}(U,\theta,t)$ = Time series of effective forces induced by wind with speed U from direction θ, acting at center of mass of floor k in direction q (= x, y, and ϑ)

$F_{kq,p}^{\mathrm{ESWL}}(U,\theta)$ = Equivalent static wind load (ESWL) in wind loading case p (WLC$_p$), induced by wind with speed U from direction θ, acting at center of mass of floor k in direction q (= x, y, and ϑ)

$f_m(U,\theta,t)$ = Time series of internal force induced at member cross section m by the effective wind forces $F_{kq}(U,\theta,t)$ ($q = x, y$, and ϑ) for given values of U and θ:

$$f_m(U,\theta,t) = \sum_{k=1}^{n} r_{mk,x} F_{kx}(U,\theta,t) + \sum_{k=1}^{n} r_{mk,y} F_{ky}(U,\theta,t) + \sum_{k=1}^{n} r_{mk,\vartheta} F_{k\vartheta}(U,\theta,t)$$

$f_{m,p}^{\mathrm{ESWL}}(U,\theta)$ = Internal force induced at member cross section m by the ESWLs in wind loading case p (WLC$_p$) for given values of U and θ:

$$f_{m,p}^{\mathrm{ESWL}}(U,\theta) = \sum_{k=1}^{n} r_{mk,x} F_{kx,p}^{\mathrm{ESWL}}(U,\theta) + \sum_{k=1}^{n} r_{mk,y} F_{ky,p}^{\mathrm{ESWL}}(U,\theta) + \sum_{k=1}^{n} r_{mk,\vartheta} F_{k\vartheta,p}^{\mathrm{ESWL}}(U,\theta)$$

h = Floor height

k = Floor level (k = 1, 2, …, n_f for n_f-floor building)

m = Index identifying member cross section

$\bar{N}$ = Mean recurrence interval (MRI) corresponding to the ordinate $P = 1 - 1/(\lambda \bar{N})$ of the Cumulative Distribution Function (CDF) fitted to the vector components $\{R_{mi}^{\mathrm{pk}}\}$ (i = number of storms)

n_{pit} = Number of the selected peaks of a time series used in the MPIT (Multiple Points-In-Time) approach

p	= Index identifying equivalent wind loading case of WLC_p (p = 1, 2, …, p_{max}, where $p_{max} = 6n_{pit}$)
$R_m^{\mathrm{RS}}(U,\theta)$	= **Response Surface** representing the peak of the time series of the wind effect $R_m(U,\theta,t)$, as a function of U and θ (1) $R_m^{\mathrm{RS}}(U,\theta) = \max_t\left(R_m(U,\theta,t)\right)$
$R_m(U_{ij})$	= Wind effects induced by wind speed U_{ij} (1)
$[R_m(U_{ij})]$	= Matrix with entries $R_m(U_{ij})$ (1)
$\{R_{m,i}^{\mathrm{pk}}\}$	= Peak wind effect induced by the wind speeds occurring in storm i, regardless of wind direction (1): $\{R_{m,i}^{\mathrm{pk}}\} = \max_j\left(R_m(U_{ij})\right)$
$R_m^{\mathrm{pk}}(\bar{N})$	= Design peak wind effect with a specified MRI $\bar{N}$ (1)
$r_{mk,q}$	= Influence coefficients representing internal force r at cross section m, induced by unit load acting at center of mass of floor k in direction q (= x, y, and ϑ)
t	= Time
U	= Mean wind speed at top of a building
U_{ij}	= Mean wind speed at top of a building in storm event i from direction $\theta = \theta_j$, based on sample of measured directional wind speeds
$[U_{ij}]$	= Matrix with entries U_{ij}
x	= Principal axis of a building
y	= Principal axis of a building, normal to x-axis
z	= Translational vertical axis of a building, normal to x- and y-axis
λ	= Mean rate of storm arrival per year
θ	= Wind direction
ϑ	= Rotation about z-axis

Note: (1) The subscript m denotes an index of member cross section, used if the wind effect by R is a DCI

REFERENCES

[1] Boggs D, Lepage A. Wind tunnel methods. Special Publication. 2006;240:125-42.

[2] Garber J, Browne MTL, Xie J, Kumar KS. Benefits of the pressure integration technique in the design of tall buildings for wind. ICWE12 Cairns. 2007.

[3] Huang G, Chen X. Wind load effects and equivalent static wind loads of tall buildings based on synchronous pressure measurements. Engineering Structures. 2007;29:2641-53.

[4] Ho TCE, Jeong UY, Case P. Components of wind -tunnel analysis using force balance test data. Wind and Structures. 2014;18:347-73.

[5] SOM. WTC wind load estimates, outside experts for baseline structural performance Appendix D. In: NCSTAR1-2 N, editor. Baseline structural performace and aircraft impact damage analysis of the World Trade Center towers. Chicago, IL, 13 April 2004 (wtc.nist.gov), also reproduced as Appendix to NIST Technical Note 1655, "Toward a standard on the wind tunnel method" (2009) by E. Simiu, pp. A1-A7 (www.nist.gov/wind). Skidmore, Owings and Merrill LLP; 2004.

[6] Yeo D, Simiu E. High-Rise Reinforced Concrete Structures: Database-Assisted Design for Wind. Journal of Structural Engineering. 2011;137:1340-9.

[7] Simiu E, Yeo D. Advances in the design of high-rise structures by the wind tunnel procedure: Conceptual framework. Wind and Structures. 2015;21:489-503.

[8] ARUP. Building Information Modelling (BIM). Available: http://wwwarupcom/services/building_modelling2017.

[9] Park S, Yeo D. Second-Order Effects on Wind-Induced Structural Behavior of High-Rise Steel Buildings. Journal of Structural Engineering. 2018;144:04017209.

[10] ANSI/AISC. Specifications for Structural Steel Buildings. AISC 360-10. Chicago, Illinois: American Institute of Steel Construction; 2010.

[11] ASCE. Minimum design loads for buildings and other structures. ASCE 7-16. Reston, VA.: American Society of Civil Engineers; 2016.

[12] Simiu E, Pintar AL, Duthinh D, Yeo D. Wind Load Factors for Use in the Wind Tunnel Procedure. ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering. 2017;3:04017007.

[13] Diana G, Giappino S, Resta F, Zasso A. Motion effects on the aerodynamic forces for an oscillating tower through wind tunnel tests. 5th European & African Conference on Wind Engineering. 2009;Florence, Italy:53-6.

[14] Marukawa H, Kato N, Fujii K, Tamura Y. Experimental evaluation of aerodynamic damping of tall buildings. Journal of Wind Engineering and Industrial Aerodynamics. 1996;59:177-90.

[15] Yeo D. Multiple Points-In-Time Estimation of Peak Wind Effects on Structures. Journal of Structural Engineering. 2013;139:462-71.

[16] ANSI/AISC. Steel construction manual. 14th ed: American Institute of Steel Construction; 2010.

Figures

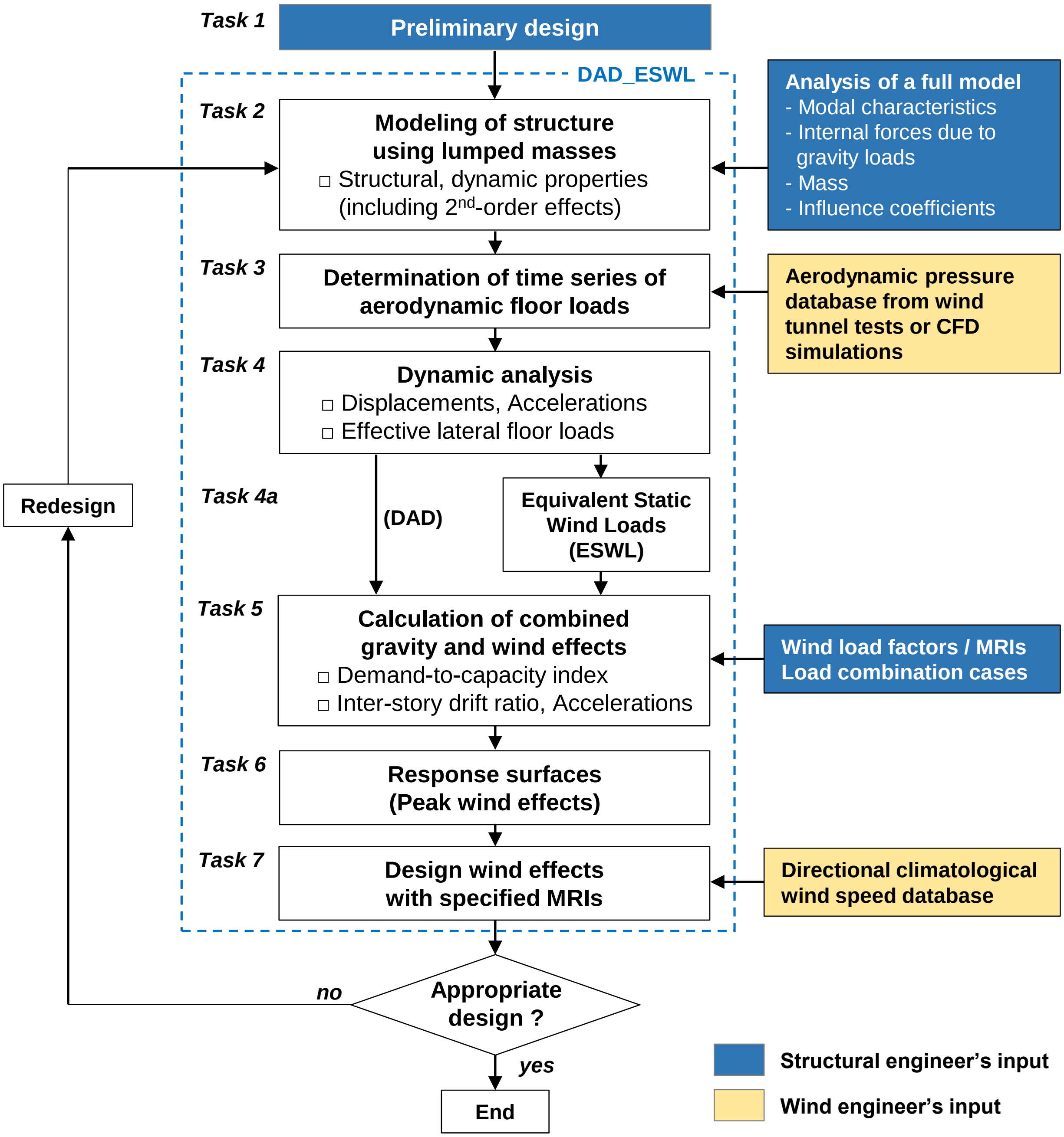


Figure 1. DAD and ESWL procedure

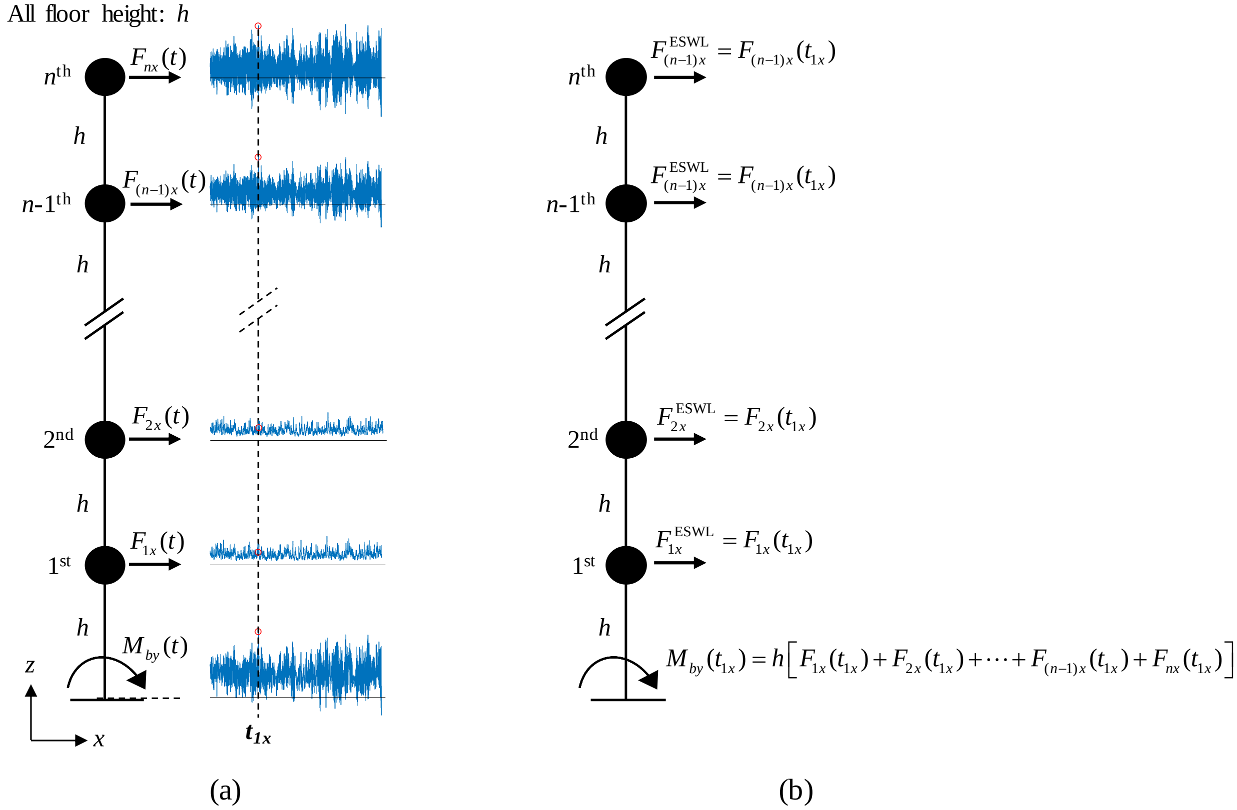


Figure 2. Lumped mass structure with (a) fluctuating wind loads in DAD and (b) equivalent static wind loads in ESWL.

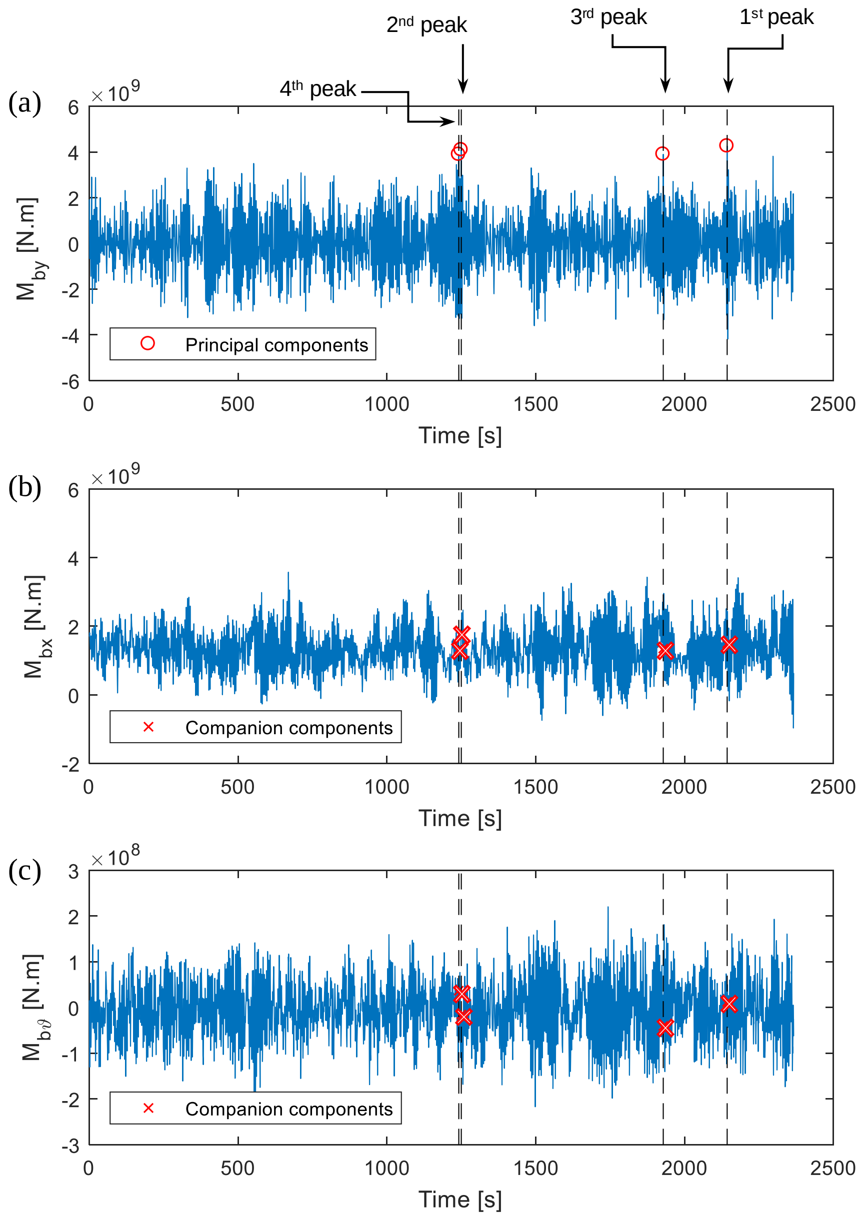
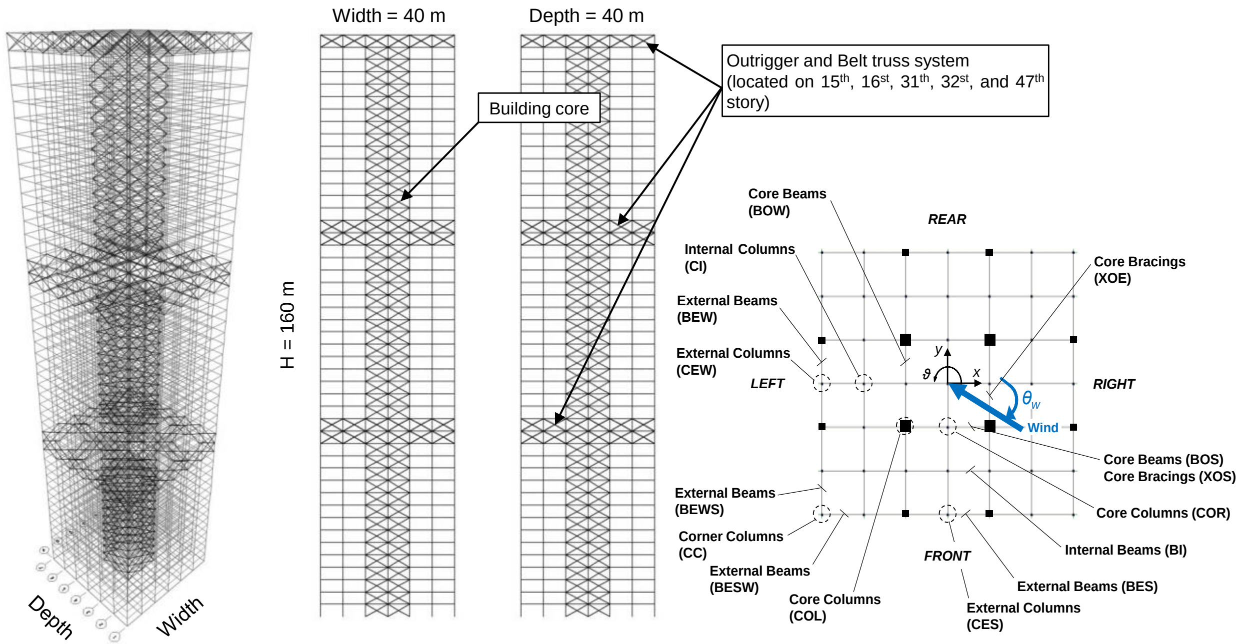


Figure 3. Sample data for three effective base moment components in wind loading case (n_{pit} = 4).



(a) 3D view (b) Front View (c) Side View (d) Selected members

Figure 4. Schematic views of structural system for the building prototype.

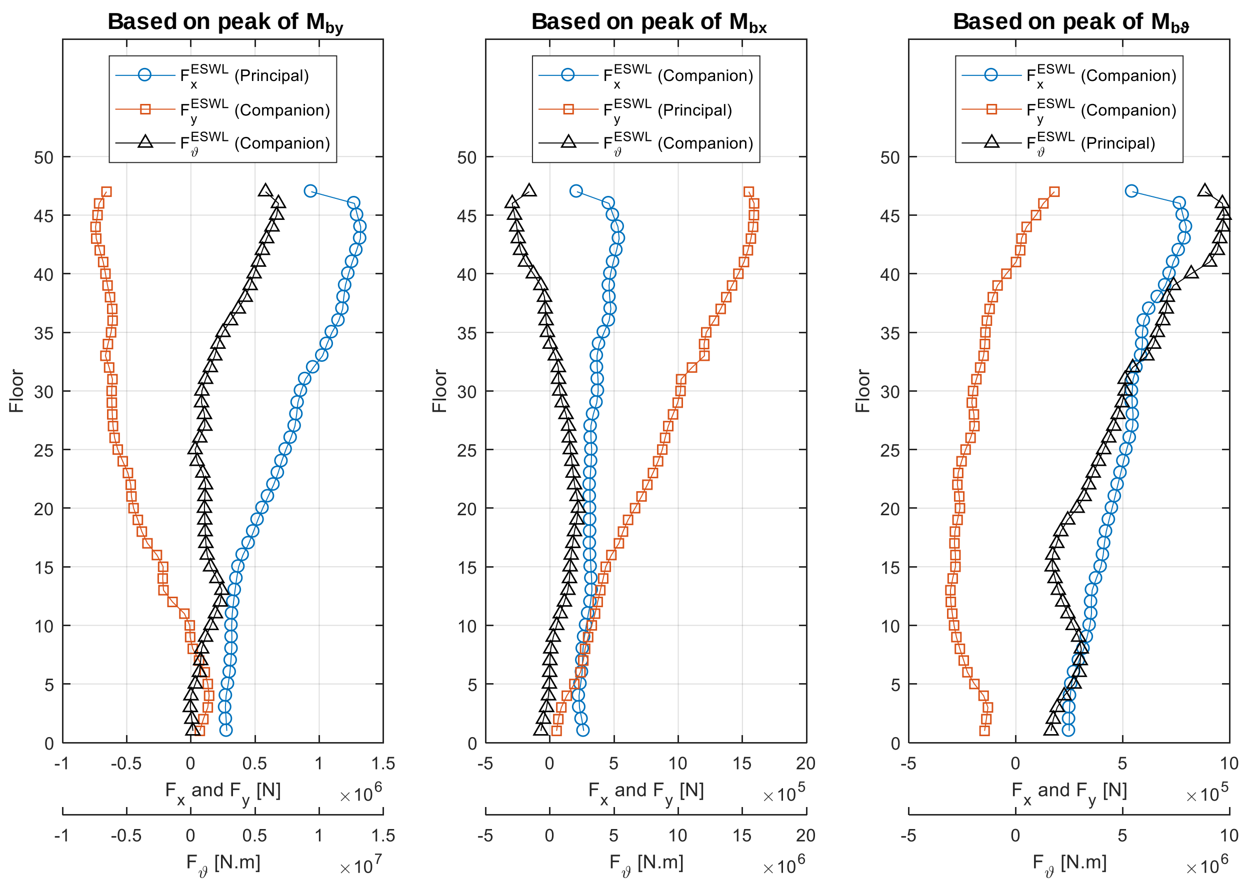


Figure 5. ESWL components based on peak of each base overturning moment $M_{by}(t)$ and $M_{bx}(t)$, and base torsion $M_{b\vartheta}(t)$ (wind direction = 180°, wind speed = 60 m/s).

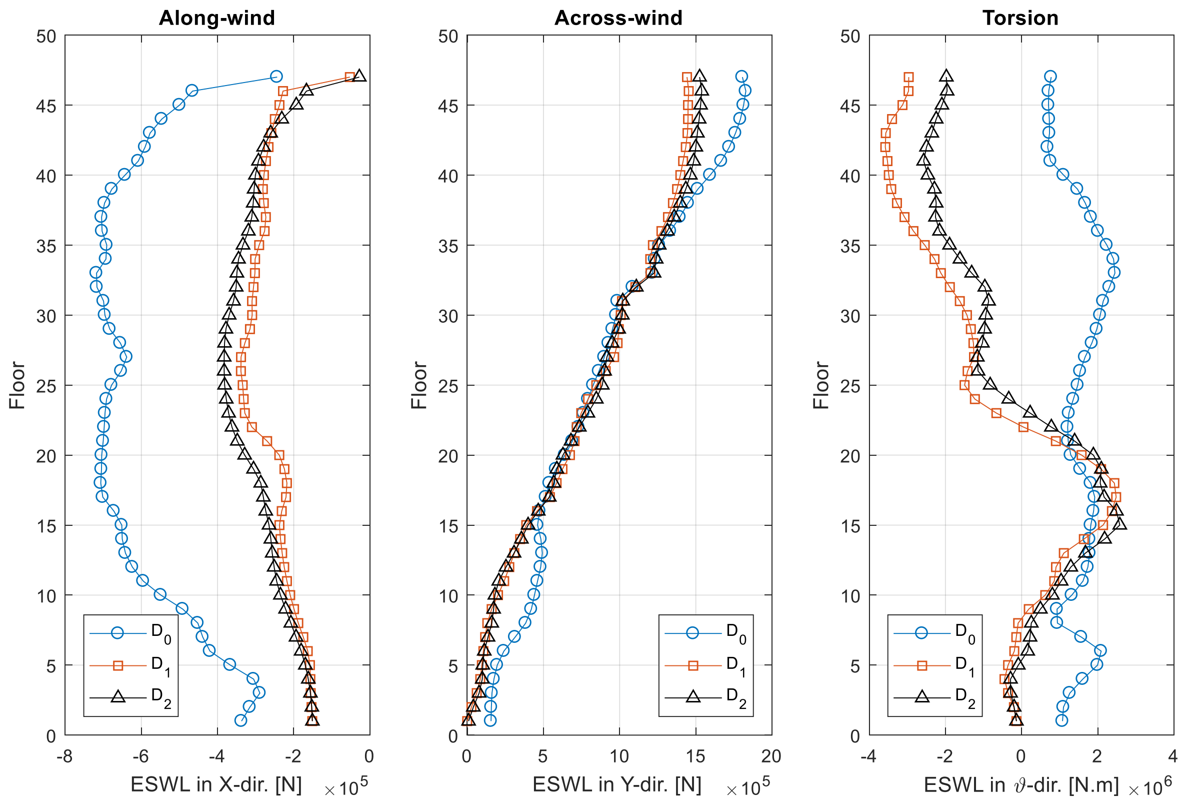


Figure 6. ESWLs determined for designs D_0, D_1, and D_2 (wind direction = 0°, wind speed = 60 m/s, wind loading case 1 out of 60 for n_{pit} = 10).

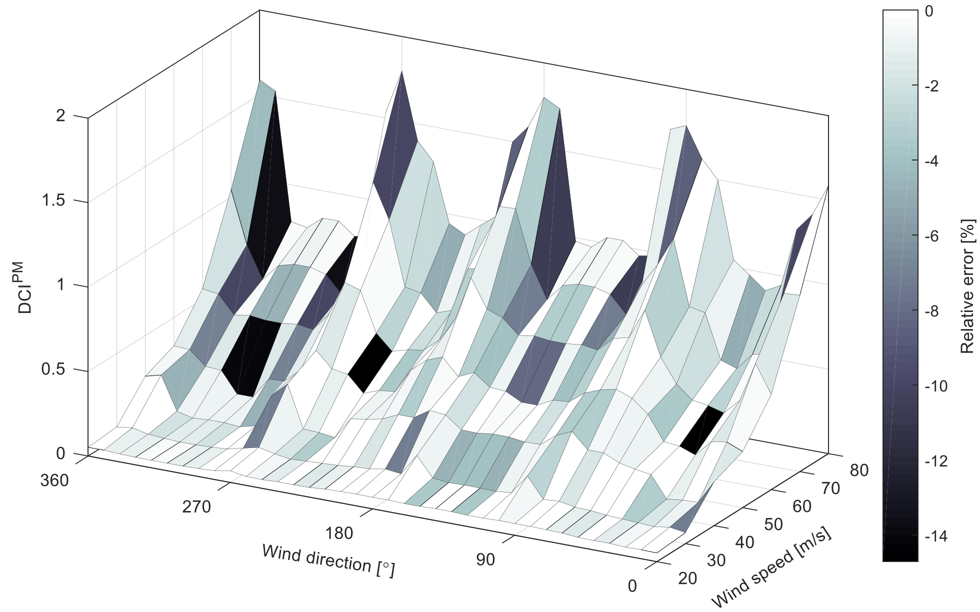


Figure 7. DCI response surface based on ESWL procedure (n_{pit} = 10, core column in 45th floor) with its deviation from that based on DAD.

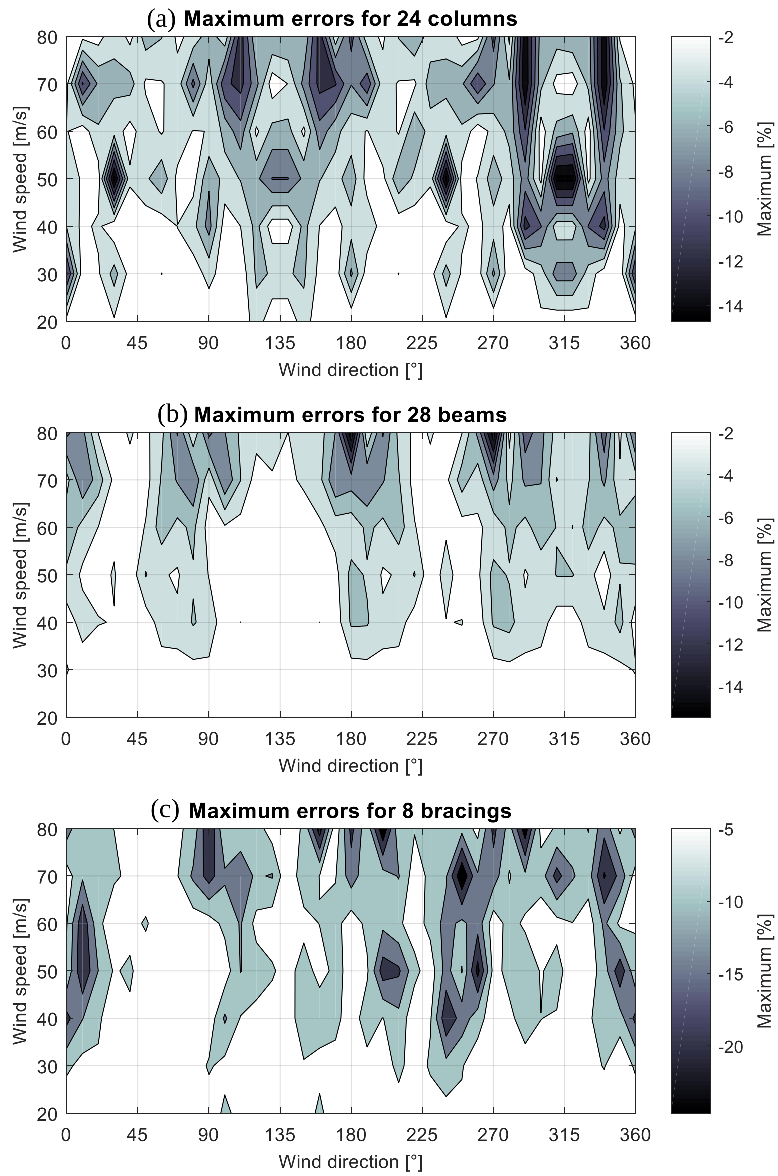


Figure 8. Contours of maximum relative errors for DCIs of (a) 24 columns, (b) 28 beams, and (c) 8 bracings with n_{pit} = 10.

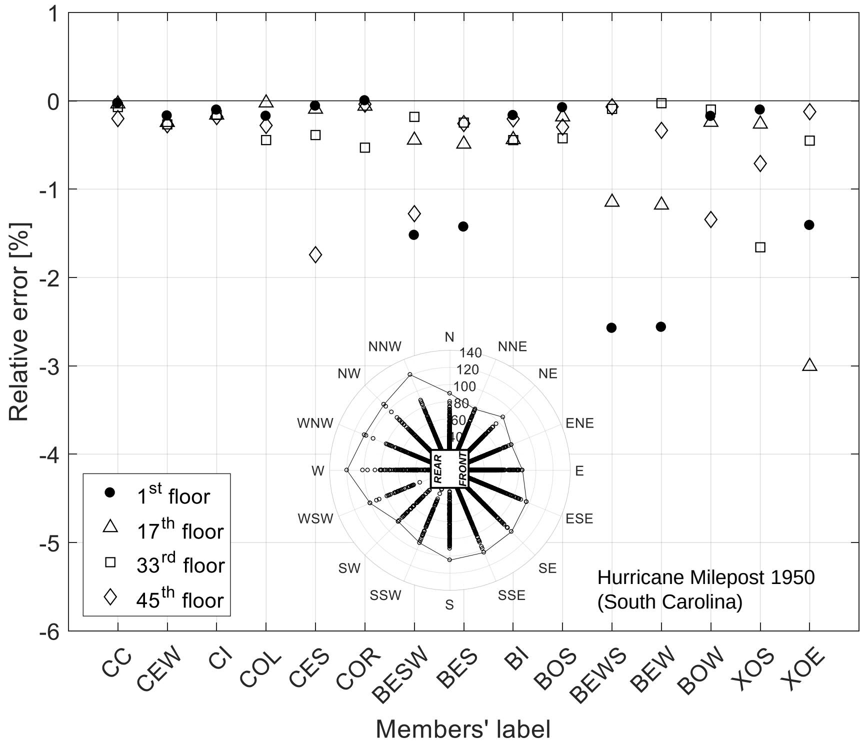


Figure 9. Relative errors of design $DCI^{pk,ESWL}$ (1700 yrs.) to design $DCI^{pk,DAD}$ (1700 yrs.), isolated building. Insert represents the extreme wind rosette at milepost 1950 (South Carolina).

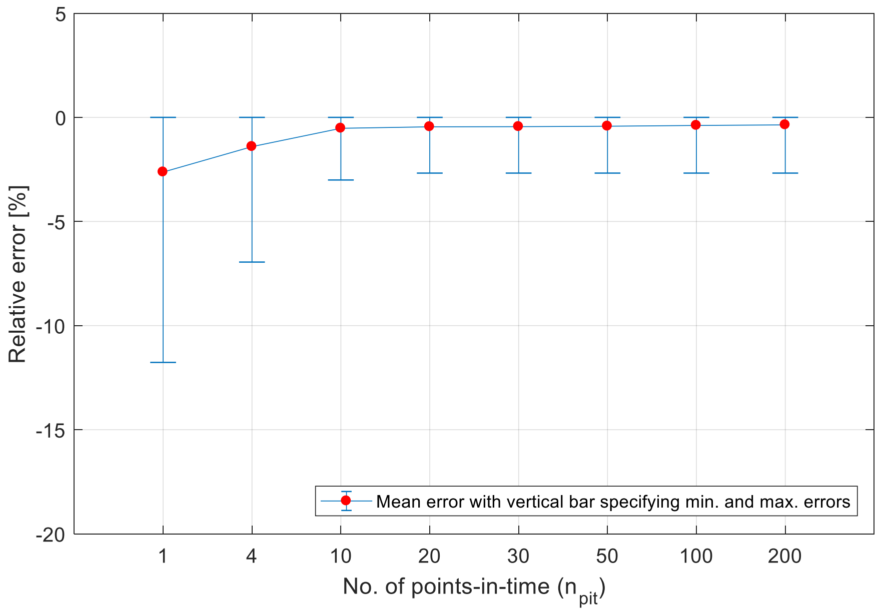


Figure 10. Statistics of relative errors of ESWL-estimated DCIs with respect to DAD counterparts as functions of n_{pit} .