Database-Assisted Design of Low-Rise Buildings for Wind Loads: Recent Developments and Comparisons with ASCE/SEI 7-05

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ABSTRACT

This paper presents recent progress in the development and implementation of the database-assisted design (DAD) methodology for low-rise buildings. The DAD methodology involves making direct use of pressure time series measured in the wind tunnel, rather than using simplified wind load cases such as those in ASCE/SEI Standard 7-05. To facilitate more widespread implementation of DAD, an aerodynamic database is being assembled at NIST. Procedures and software have been developed to automate the handling of aerodynamic data in the database, and an interpolation scheme has been developed to enable the use of DAD for cases in which a wind tunnel model with matching dimensions is not available in the database. This paper also presents comparisons of peak bending moments computed using DAD with those resulting from the simplified wind load cases in ASCE 7-05. The agreement is found to be quite good for bending moments that were considered in the development of the simplified load cases used in ASCE/SEI 7-05. However, when an intermediate location is considered, the maximum bending moment from DAD is found to be more than twice as large as that from ASCE/SEI 7-05, indicating unconservatism.

INTRODUCTION

While specialized wind tunnel tests are routinely conducted in the design of high-rise buildings, constraints on project budgets and timelines generally do not permit wind tunnel testing in the design of low-rise buildings. Instead, simplified wind load cases such as those in ASCE/SEI Standard 7-05 [1] are typically used. These simplified load cases are based on the results of wind tunnel tests conducted in the 1970s; however, reducing these test results into a manageable tabular format has necessitated considerable simplification and loss of information. Database-assisted design (DAD) (e.g., [2], [3]) is a methodology for analysis and design of structures that makes direct use of pressure time histories recorded in the wind tunnel, with a view to achieving structures that are more risk-consistent and potentially more economical. To facilitate more widespread use of the DAD approach for low-rise buildings, an aerodynamic database is being assembled at the National Institute of Standards and Technology (NIST), containing measured pressure time series for a large number of gable-roofed building models with various dimensions. As part of this database, approximately 38 building model variations have been tested to date at the University of Western Ontario (UWO) [4], and these data are to be made publicly available, along with software for using the data in structural analysis and design.

This paper presents recent progress at NIST in the development and implementation of the DAD methodology for low-rise buildings. Taking advantage of a standard archival format for wind tunnel data developed at UWO [4], procedures and software have been developed to automate the handling of aerodynamic data. The archived pressure tap coordinates are used to automatically distribute the measured pressures to the appropriate load points on the structural frames, thus eliminating the manual preprocessing required in earlier DAD software [5].

Because of practical limitations on the number of wind tunnel models that can be tested for inclusion in the NIST aerodynamic database, establishment of DAD as a broadly applicable methodology requires a reliable means of interpolation, to enable prediction of structural responses for cases in which the building dimensions do not precisely match the dimensions of an available model in the database. Interpolation schemes proposed previously have aimed at interpolation of pressure time series between those measured on different models [6], and artificial neural networks have been used in an effort to capture the complex nonlinear dependence of the measured pressure coefficients on building geometry, wind direction, and tap location [7]. This paper describes an alternative approach for interpolation, in which peak structural responses (e.g., bending moments and axial forces) are interpolated rather than pressure time series, thus eliminating the necessity of explicitly accounting for spatial and temporal correlations in the interpolation scheme. An overview of the proposed methodology is first presented, and then several interpolation test cases are presented, which indicate that the proposed methodology can give quite accurate predictions of peak structural responses.

In addition to its potential for direct application in design, the DAD methodology also holds promise to facilitate assessment and improvement of current wind load standards such as those in ASCE/SEI 7-05. Ho et al. [8] have previously presented comparisons of structural responses computed directly from wind tunnel data with those resulting from wind load provisions in several different design standards. In this paper, peak bending moments computed using DAD for a particular structural frame are compared with those resulting from the simplified wind load cases in ASCE/SEI 7-05. The agreement is found to be quite good for bending moments at the knee and at the ridge of the frame, which were considered in the development of the "pseudo pressure coefficients" used in ASCE/SEI 7-05. However, when the bending moment at an intermediate location is considered, the maximum moment from DAD is found to be more than twice as large as that from ASCE/SEI 7-05, indicating unconservatism. While this paper presents a comparison for just one particular structural system, the current DAD software, known as *wind*PRESSURE, enables such comparisons for a wide range of building geometries and structural configurations and is available for download at www.nist.gov/wind.

SUMMARY OF DATABASE-ASSISTED DESIGN PROCEDURES

Database-assisted design makes direct use of pressure time series recorded simultaneously at a large number of pressure taps distributed over the surface of a scale building model, as illustrated in Figure 1. Building models are tested in boundary layer wind tunnels, which enable simulation of the turbulent flow characteristics and the variation in mean flow velocity with elevation that are associated with specified terrain conditions, and tests are conducted over a range of different values of the wind direction θ (see Figure 1). Pressure time series are typically stored in nondimensional form as pressure coefficients C_p, from which pressures corresponding to any particular wind speed can be obtained by scaling as follows:

$$p = \tfrac{1}{2}\rho V_{H,1\mathrm{h}}^2 C_p \qquad (1)$$

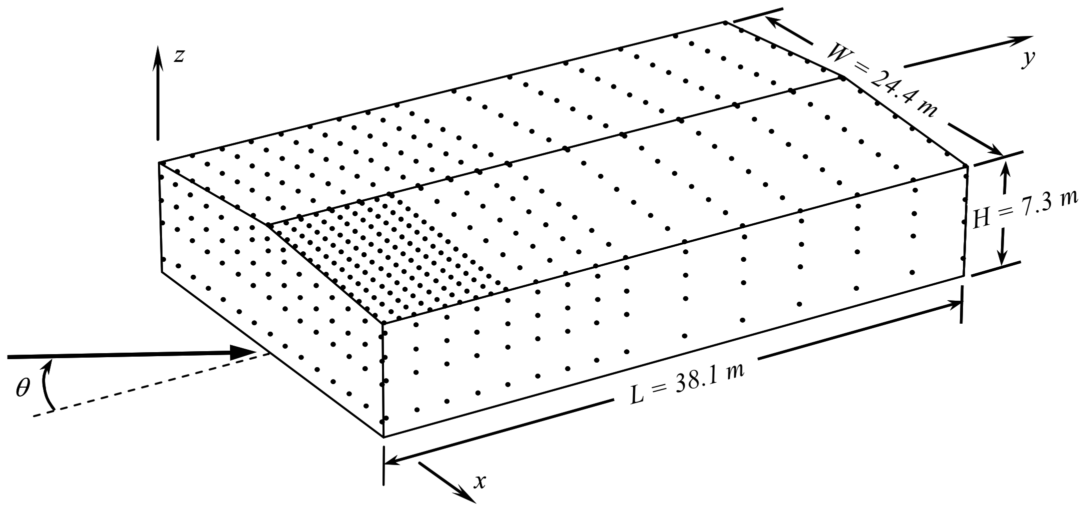

FIGURE 1

PERSPECTIVE VIEW OF WIND TUNNEL MODEL SHOWING TAP LOCATIONS, WIND DIRECTION θ, AND BUILDING DIMENSIONS (1:12 ROOF SLOPE)

where $\rho = 1.225\,\mathrm{kg/m^3}$ is the air density, and $V_{H,1\mathrm{h}}$ is the hourly averaged wind speed at eave height.

The first stage in DAD involves transforming the wind pressures, which are applied to the cladding of the building, to resultant wind forces, which are applied to the primary structural system at a number of discrete locations. A typical structural system for a low-rise metal building, as depicted in Figure 2, consists of structural frames spanning the width of the building, with girts and purlins spanning between the frames and supporting the wall panels and roof panels, respectively. In structural design, the frames are typically analyzed independently, with resultant wind loads applied to the frames at the girt and purlin attachment locations. As described in [3], the distribution of the wind pressures from the tap locations to the girt and purlin locations can be represented using a "tributary matrix," which is assembled automatically within the *wind*PRESSURE software by analyzing each roof and wall panel in turn and using the pressure tap coordinates stored in the aerodynamic database. Using the resulting "tributary matrix," the recorded time series of wind pressures for each wind direction are transformed into time series of resultant loads at the girt and purlin attachment locations.

Once time series of resultant wind loads on the primary structural system have been obtained, time series of the structural responses of interest (e.g., bending moments, shear forces, and displacements) can be evaluated. This is conveniently accomplished using influence coefficients, which give the values of the structural responses of interest resulting from unit forces at each loading point. Influence coefficients can be obtained using standard structural analysis software by defining load cases corresponding to unit forces at each loading point and computing the resulting wind effects of interest. These influence coefficients can be supplied to the *wind*PRESSURE software in a spreadsheet-format input file. Alternatively, the *wind*PRESSURE software incorporates a linear frame element modeling capability for internal evaluation of influence coefficients from specified section properties.

Peak structural responses (i.e., maximum and minimum values) are of the greatest interest in structural design, and therefore it is necessary to evaluate peaks from the time series of structural responses computed as discussed previously. While the maximum and minimum values of a time series are readily evaluated, these *observed peaks* can exhibit wide variability from one realization to another, due to the highly fluctuating nature of wind pressures (i.e., significant differences might be expected in the peak wind effects computed using several different sets of pressure time series obtained under nominally identical conditions in the same wind tunnel). Therefore it is generally preferable to use a more stable estimator for the expected peaks. Sadek and Simiu [9] present a procedure for estimating peaks that involves evaluation of probability distributions for the peaks through extension of classical results for Gaussian processes. As part of the software development in this study, a modified version of this procedure has been implemented within the *wind*PRESSURE software. The examples in this paper present expected peaks, computed using this approach, rather than observed peaks.

The building dimensions and tap coordinates illustrated in Figure 1 correspond to a 1:200 scale model with a 1:12 roof slope tested by Ho et al. [4], and the examples presented in this paper use pressure time series obtained from this model for "open country" terrain. The examples also consider the structural system depicted in Figure 2, which corresponds to the same full-scale dimensions as the model shown in Figure 1 and consists of six frames equally spaced at 7.6 m (25 ft) intervals. The first interior frame, highlighted in Figure 2, is selected for analysis in the examples. The column bases are pinned, and preliminary member sizes were determined using wind loads from ASCE/SEI 7-05 for a building in open country terrain in Miami. Bending moments at three cross-sections are considered, and the influence coefficients corresponding to each of these bending moments are shown in Figure 3. Figure 4 shows peak values of each of these structural responses as a function of wind direction θ, computed using the database-assisted design procedures outlined in this section. Peak responses are presented for a unit wind speed from each direction ($V_{H,1\mathrm{h}} = 1\ \mathrm{m/s}$). These results are further discussed in the following sections, in which they are compared with responses computed using ASCE/SEI 7-05 and with responses estimated by interpolation from models with different dimensions.

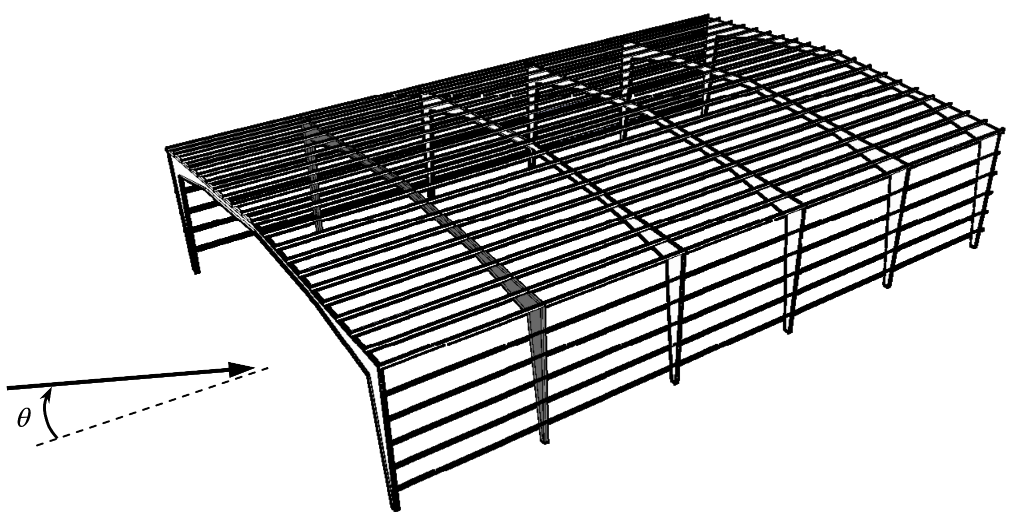

FIGURE 2

PERSPECTIVE VIEW OF STRUCTURAL SYSTEM SHOWING FRAMES, GIRTS, AND PURLINS (FIRST INTERIOR FRAME HIGHLIGHTED)

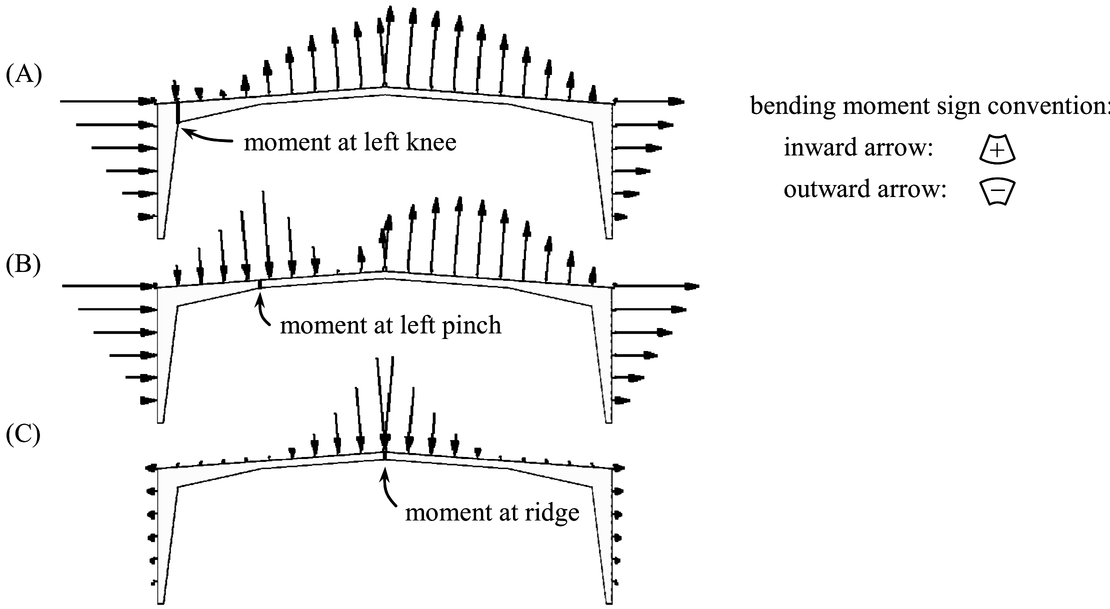

FIGURE 3

INFLUENCE COEFFICIENTS ASSOCIATED WITH (A) MOMENT AT LEFT KNEE, (B) MOMENT AT LEFT PINCH, AND (C) MOMENT AT RIDGE. VECTORS SHOW BENDING MOMENTS RESULTING FROM A UNIT FORCE *INWARD* AT EACH GIRT/PURLIN LOCATION.

COMPARISONS WITH ASCE/SEI 7-05

The peak bending moments from ASCE/SEI 7-05 shown in Figure 4 were calculated using the Analytical Procedure (Method 2) for low-rise buildings [1, §6.5]. Only external pressures were considered in the DAD computations, and therefore, internal pressures were not included in the wind loads from ASCE/SEI 7-05 for consistency. The directionality factor K_d was set to unity, because this factor is intended to represent climatological effects (i.e., the reduced probability of the highest wind speed coming from the most unfavorable wind direction), which are not considered in the present study. With $K_d = 1$, the peak bending moment computed from ASCE/SEI 7-05 should nominally correspond to the worst-case bending moment from DAD over all wind directions. The importance factor I and the topographic factor K_{zt} were also set to unity for comparison with the DAD results. With these simplifications, the external wind pressures from Eq. (6-18) of ASCE/SEI 7-05 [1] can be expressed as follows:

$$p = \tfrac{1}{2}\rho K_h V_{10\mathrm{m},3\mathrm{s}}^2 (GC_{pf}) \tag{2}$$

where the velocity pressure exposure coefficient $K_h = 2.01(7.3/274)^{(2/9.5)} = 0.937$ for an eave height of $H = 7.3\ \mathrm{m}$ over open terrain, the pseudo external pressure coefficients GC_{pf} for different zones of the roof and walls are given in Figure 6-10 of Ref. [1], and $V_{10\mathrm{m},3\mathrm{s}}$ is a $3\ \mathrm{s}$ gust wind speed at $10\ \mathrm{m}$ elevation over open terrain. An hourly averaged wind speed of $V_{H,1\mathrm{h}} = 1\ \mathrm{m/s}$ at eave height was considered in the DAD computations, and a corresponding value of $V_{10\mathrm{m},3\mathrm{s}} = 1.596\ \mathrm{m/s}$ for use in (2) can be obtained by using Figure C6-4 of Ref. [1] to account for the influence of averaging time ($V_{3\mathrm{s}} / V_{1\mathrm{h}} \cong 1.51$) and assuming a logarithmic mean wind speed profile with a roughness length of $z_0 = 0.03\ \mathrm{m}$ ($V_{10\mathrm{m}} / V_{7.3\mathrm{m}} \cong 1.06$).

Comparison of the peak bending moments from DAD and from ASCE/SEI 7-05 in Figure 4 shows quite good agreement for worst-case bending moments at the left knee and at the ridge. However, for the bending moment at the left pinch, the worst-case bending moment from DAD is more than twice as large as the peak bending moment computed using ASCE/SEI 7-05, indicating that ASCE/SEI 7-05 is unconservative in this case. The Commentary of ASCE/SEI 7-05 [1, §C6.5.11] notes that in the development of the pseudo pressure coefficients GC_{pf}, only bending moments at the knees and at the ridge were considered, in addition to total uplift and total horizontal shear. Consequently, we find for this particular structure that significant discrepancies can be observed when bending moments at other locations are considered. By more faithfully representing the wind-induced internal forces at any cross-section within the structure, the DAD approach can be expected to lead to more risk-consistent structures.

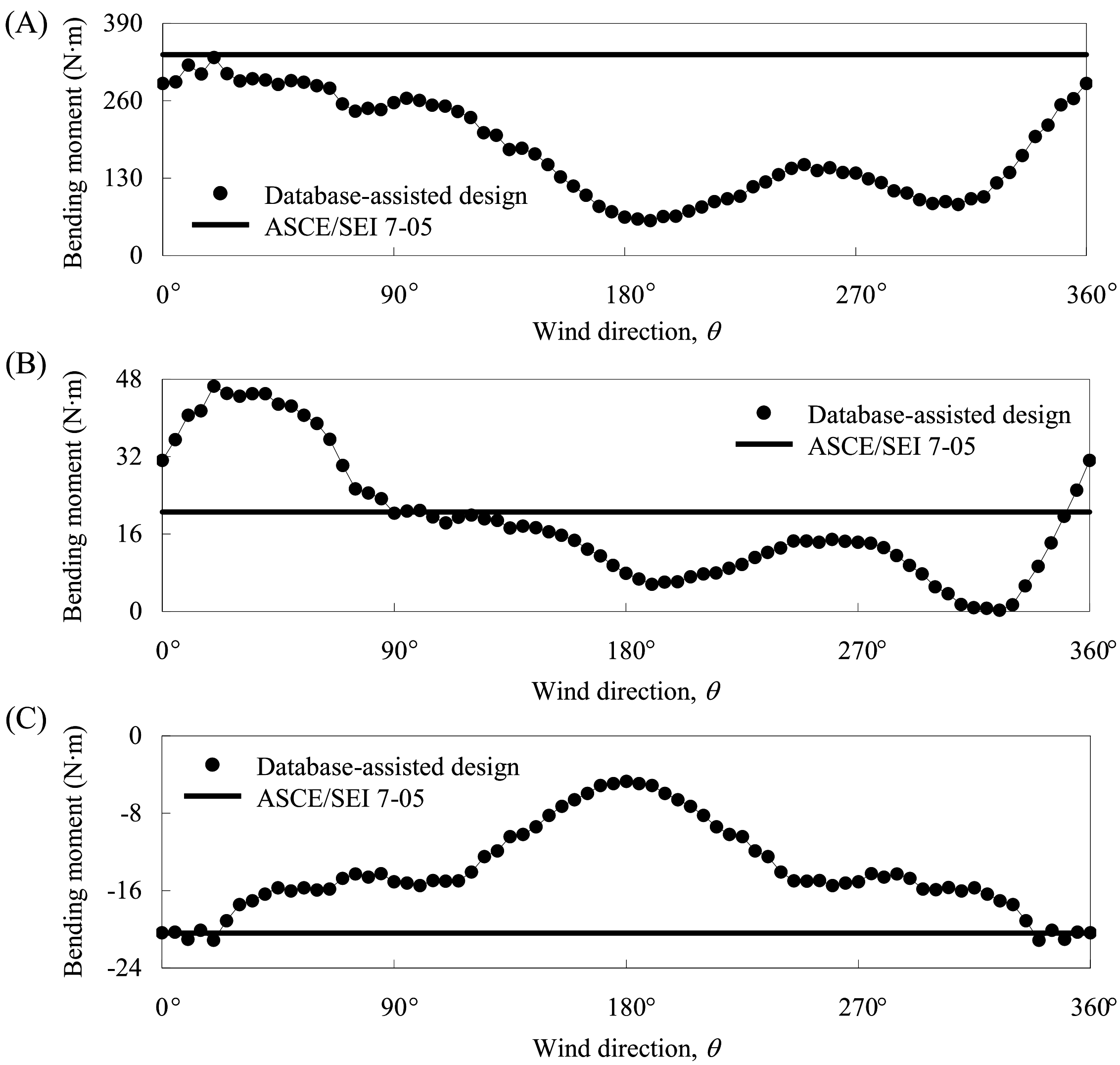

FIGURE 4

PEAK BENDING MOMENTS IN THE FIRST INTERIOR FRAME COMPUTED FROM WIND TUNNEL PRESSURES FOR A UNIT WIND SPEED FROM EACH WIND DIRECTION θ (HOURLY AVERAGED WIND SPEED OF 1 m/s AT EAVE HEIGHT). PEAK BENDING MOMENTS COMPUTED FROM ASCE/SEI 7-05 ARE ALSO SHOWN FOR COMPARISON. (A) MAXIMUM MOMENT AT LEFT KNEE; (B) MAXIMUM MOMENT AT LEFT PINCH; (C) MINIMUM MOMENT AT RIDGE.

INTERPOLATION PROCEDURE AND TEST CASES

For cases in which a building model with matching dimensions is unavailable in the aerodynamic database, the proposed interpolation methodology can be summarized as follows:

1. One or more building models are selected, whose dimensions most closely match the dimensions of the building of interest. A vector $\mathbf{d}_i$ is introduced to represent the geometry of the ith building model:
$$\mathbf{d}_i = \begin{bmatrix} H_i / z_{0i} & L_i / W_i & H_i / W_i & \beta_i \end{bmatrix}^T \tag{3}$$
where $\beta_i = \tan^{-1}(2R_i / W_i)$ is the roof slope (in degrees); W_i, L_i, H_i, and R_i are shown in Figure 1; and z_{0i} is the terrain roughness length. The index $i = 0$ is used to denote the dimensions of the building of interest, and a vector of deviations in the dimensions of the ith model can then be defined as
$$\mathbf{\Delta d}_i = \mathbf{d}_i - \mathbf{d}_0 \tag{4}$$
The components of $\mathbf{\Delta d}_i$ can be scaled by empirically determined factors that represent the sensitivity of building aerodynamics to changes in each component:
$$\mathbf{\Delta \tilde{d}}_i = \mathbf{S}\mathbf{\Delta d}_i \quad \text{where} \quad \mathbf{S} = \mathrm{diag}(S_1, S_2, S_3, S_4) \tag{5}$$
The scaled deviation vectors $\mathbf{\Delta \tilde{d}}_i$ are then used in selecting the building models whose dimensions match most closely, as well as in the interpolation of step 3.
2. For each of the selected building models, the archived pressure time series are loaded. The pressure time series are used in conjunction with influence coefficients for the structural responses of interest to estimate peak values of the structural responses for a unit wind speed from each direction at eave height. These peak responses are called Directional Influence Factors (DIFs) and can be represented as a matrix $\mathbf{X}_i^{peak}$, where the rows correspond to different responses, the columns correspond to different wind directions, and the subscript $i = 1, 2, \ldots, b$ denotes the number of the building model. In transforming the measured pressures to structural loads, the tap coordinates are scaled to match the dimensions of the structure of interest and the pressures are distributed accordingly. With the coordinate system as shown in Figure 1, the scaled tap coordinates $(\tilde{x}, \tilde{y}, \tilde{z})$ are obtained from the original tap coordinates (x, y, z) as follows:
$$\tilde{x} = \frac{W_0}{W_i} x \,; \; \tilde{y} = \frac{L_0}{L_i} y \,; \; \tilde{z} = \begin{cases} (H_0 / H_i) z & , \; z \le H_i \\ H_0 + (R_0 / R_i)(z - H_i), & z > H_i \end{cases} \tag{6}$$
3. A best estimate of the DIF matrix for the building of interest is then obtained as a weighted average of the DIF matrices from the different building models as follows:
$$\tilde{\mathbf{X}}^{peak} = \sum_{i=1}^{b} \gamma_i \mathbf{X}_i^{peak} \quad \text{where} \quad \gamma_i = \left(\sum_{k=1}^{b} \frac{\left\| \mathbf{\Delta \tilde{d}}_i \right\|}{\left\| \mathbf{\Delta \tilde{d}}_k \right\|} \right)^{-1} \tag{7}$$
The interpolation scheme of (7) gives greater weight to models whose dimensions more closely match the building of interest, and it is noted that for one-dimensional interpolation with two building models ($b = 2$), (7) reduces to simple linear interpolation.

To evaluate the accuracy of the proposed interpolation methodology, pressure data are used from nine different building model variations tested at UWO, with dimensions listed in Table 1.

Using these data, four interpolation test cases are considered, as listed in Table 2. In each test case, the goal is to predict peak structural responses for a building with the full-scale dimensions of model 0, given in the first row of Table 1, using pressure data from two models with different dimensions. The full-scale dimensions of model 0 are the same as those illustrated previously in Figure 1. Because these interpolation test cases consider differences in only one component of $\mathbf{d}_i$ at a time (or two components varied according to a fixed ratio in case D), the relative magnitude of the sensitivity factors in (5) has no influence on the interpolation of (7), and $\mathbf{S}$ can simply be set to the identity matrix in evaluating the scaled deviation vectors, so that $\mathbf{\Delta\tilde{d}}_i = \mathbf{\Delta d}_i$. These test cases can be used to assess appropriate values of the sensitivity factors $\mathbf{S}$ to be used for more complex cases of interpolation, and efforts along these lines are currently in progress.

In evaluating structural responses, a structural system consisting of six frames equally spaced at 7.6 m (25 ft) intervals is considered, as shown in Figure 2. The structural response of interest is the bending moment at the left knee, for which the influence coefficients were shown previously in Figure 3(a). Figure 5 shows maximum values of this bending moment plotted against the wind direction θ, defined as shown in Figure 2. In each interpolation test case, errors can be assessed by comparing the interpolated value with the "true" value computed using pressure data from model 0. The maximum and root-mean-square (RMS) errors over all wind directions are presented in Table 2 for each case. Figure 5 and Table 2 show that the interpolated peak bending moments are quite accurate in general, being best in case D (models with different eave heights) and worst in case C (models with different roof slopes). In all cases the maximum errors are less than 15 %, and the RMS errors are less than 6 %.

Model number, i	Width, W_i	Length, L_i	Eave Height, H_i	Roof Slope, β_i
0	24.4 m (80 ft)	38.1 m (125 ft)	7.3 m (24 ft)	4.8° (1:12)
1	12.2 m (40 ft)	19.1 m (62.5 ft)	3.7 m (12 ft)	4.8° (1:12)
2	36.6 m (120 ft)	57.2 m (187.5 ft)	12.2 m (40 ft)	4.8° (1:12)
3	12.2 m (40 ft)	19.1 m (62.5 ft)	7.3 m (24 ft)	4.8° (1:12)
4	36.6 m (120 ft)	57.2 m (187.5 ft)	7.3 m (24 ft)	4.8° (1:12)
5	24.4 m (80 ft)	38.1 m (125 ft)	7.3 m (24 ft)	1.2° (¼:12)
6	24.4 m (80 ft)	38.1 m (125 ft)	7.3 m (24 ft)	14° (3:12)
7	24.4 m (80 ft)	38.1 m (125 ft)	4.9 m (16 ft)	4.8° (1:12)
8	24.4 m (80 ft)	38.1 m (125 ft)	9.8 m (32 ft)	4.8° (1:12)

TABLE 1
FULL-SCALE DIMENSIONS OF MODELS USED IN INTERPOLATION TEST CASES (1:200 SCALE).

Interpolation test case	Selected models	Maximum error	RMS error
A	1, 2	28 N·m (8.3 %)	9.1 N·m (2.7 %)
B	3, 4	45 N·m (13 %)	16 N·m (4.7 %)
C	5, 6	47 N·m (14 %)	19 N·m (5.6 %)
D	7, 8	20 N·m (6.2 %)	6.1 N·m (1.8 %)

TABLE 2
INTERPOLATION TEST CASES AND CORRESPONDING ERRORS IN INTERPOLATED BENDING MOMENT. (ERROR PERCENTAGES ARE REFERENCED TO THE MAXIMUM BENDING MOMENT COMPUTED USING THE "TRUE" PRESSURE DATA FROM MODEL 0, WHICH IS 333 N·m.)

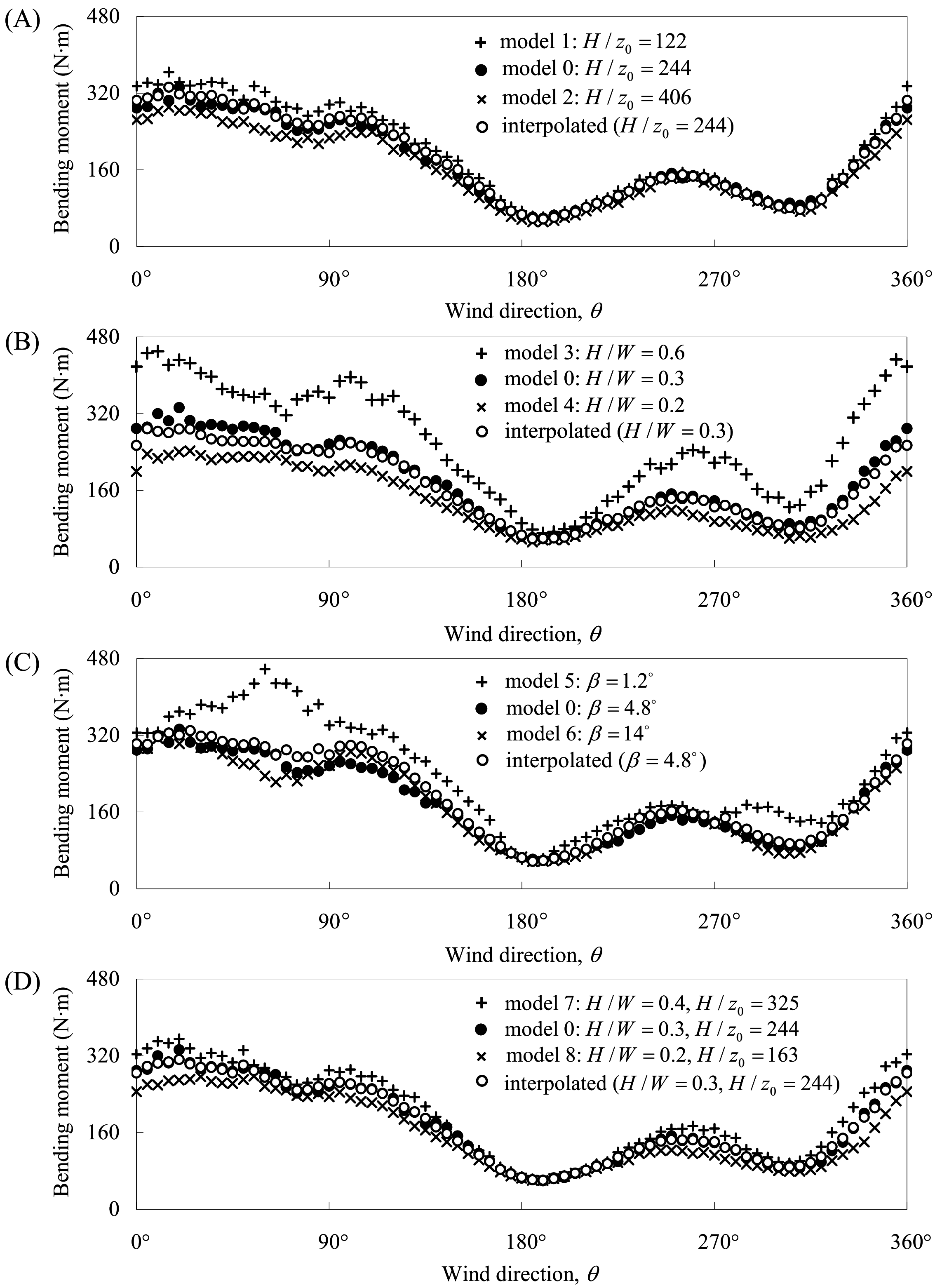

FIGURE 5

MAXIMUM VALUES OF BENDING MOMENT AT THE LEFT KNEE OF THE FIRST INTERIOR FRAME FOR THE FOUR INTERPOLATION TEST CASES OF TABLE 2: (A) CASE A; (B) CASE B; (C) CASE C; (D) CASE D. (VALUES CORRESPOND TO AN HOURLY AVERAGED WIND SPEED OF 1 m/s AT EAVE HEIGHT.)

CONCLUDING REMARKS

This paper has summarized recent progress at NIST in the development and implementation of the DAD methodology for low-rise buildings. Comparisons of peak bending moments computed using DAD with those computed using ASCE/SEI 7-05 have shown that discrepancies can be significant for cross-sections that were not considered in the development of the "pseudo pressure coefficients" used in the standard. By more faithfully representing the wind-induced internal forces at any cross-section, the DAD approach can be expected to lead to more risk-consistent structures. Further, more comprehensive comparisons should be conducted for different building dimensions and different structural responses. Such comparisons will be facilitated by the extensive automation that has been incorporated into the *wind*PRESSURE DAD software. A relatively simple multidimensional interpolation scheme has also been presented, which enables prediction of structural responses for cases in which a wind tunnel model with matching dimensions is unavailable. Four test cases were presented, which illustrated that the interpolation scheme can give quite accurate results. In all cases considered, the maximum errors were less than 15 %, and the RMS errors were less than 6 %. Efforts are currently underway to test the interpolation scheme for deviations in multiple building dimensions and to establish bounds on the applicability of the proposed interpolation scheme.

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