Wind Effects on Low-Rise Buildings: Database-Assisted Design vs. ASCE 7-05 Standard Estimates

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Abstract: Peak bending moments are compared for a set of steel portal frames of industrial buildings in open terrain, calculated using database-assisted design (DAD) techniques and ASCE 7-05 Standard plots. The comparisons indicate that, depending on building dimensions, the peak bending moments at the knee based on DAD techniques are generally larger by 10 % to 40 % than their counterparts based on the ASCE 7-05 plots. (In one case with a relatively steep roof slope of 26.6 ° the discrepancies exceed 90 %.) Discrepancies increase with increasing roof slope and with increasing eave height.

CE Database subject headings: Aerodynamics; Buildings, low-rise; Databases; Structural design; Wind forces; Wind tunnel tests.

Introduction

This note compares peak bending moments in steel portal frames of industrial buildings in open terrain calculated using database-assisted design (DAD) techniques on the one hand and by using the Analytical Procedure from American Society of Civil Engineers (ASCE) Standard 7-05 (ASCE 2006, §6.5) on the other. DAD is a methodology for

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analysis and design of structures that makes direct use of pressure time histories measured in the wind tunnel (e.g., Whalen et al. 1998, Rigato et al. 2001, Simiu et al. 2003, Main and Fritz 2006). The aerodynamic database used in this study was developed by the University of Western Ontario (UWO, see Ho et al. 2005). The ASCE 7-05 Analytical Procedure entails the use of simplified coefficients, referred to in the Commentary of the Standard (§ C6.5.11 and Fig. C6-6) as “pseudo-pressure” coefficients, and based on wind tunnel data measured at UWO mostly in the 1970s (Davenport et al. 1979). The “pseudo-pressure” coefficients were developed with the aim of enveloping peak values of bending moments at the knees and ridge (see Fig. 1a), resultant vertical uplift and horizontal shear for a total of about 15 distinct building geometries.

St. Pierre et al. (2005) compared bending moments, vertical uplift, and horizontal shear derived from pressures measured by Ho et al. (2005) with corresponding values computed using ASCE 7-02 plots. They noted that the responses predicted by ASCE 7-02 in many cases underestimated the responses obtained using the recent pressure measurements. These discrepancies were attributed largely to the lower turbulence intensities in the earlier experiments. Additional sources of discrepancy are that the 1970s UWO tests were conducted predominantly for wind directions in increments of 45°, as opposed to 5° in the later tests, with the number of pressure taps in the earlier tests lower by almost one order of magnitude. Also, in developing “pseudo-pressure” coefficients, the distance between frames and the structural properties of the frames had specified values, whereas in the Standard the coefficients are assumed to be valid regardless of those values. Most importantly, the values of the “pseudo-pressure” coefficients were obtained by eye, rather than by systematic and rigorous calculation. Even where the coefficients result in reasonably correct values of the bending moments at the knees and ridge, their suitability for calculating bending moments at other locations is not guaranteed. In fact comparisons by Main (2006a) of bending moments resulting from the pressure measurements of Ho et al. (2005) with those predicted by ASCE 7-05 showed that the ASCE 7-05 loads significantly under-predicted the bending moments at the "pinch" (see Fig. 1a), even for a case in which fairly good agreement was observed for the moments at the knee and the ridge.

In the present study, seven buildings were selected for analysis, with dimensions listed in Table 1. All of the buildings were of gable-roofed geometry, as illustrated in Fig. 2. Four of the selected buildings had the same roof slope (θ = 14°), with eave heights H varying from 4.9 m to 12.2 m (16 ft to 40 ft). The remaining three buildings were selected to cover a range of available roof slopes in the data set from Ho et al. (2005). This represents a wider range of roof slopes than was considered in the comparisons of St. Pierre et al. (2005), who considered two roof slopes (θ = 4.8° and θ = 14°), each with four different eave heights. All buildings are low-rise structures with H < 18.3 m (60 ft), and the structural frames were designed for ASCE 7-05 Exposure C ("open country") at a location near Miami, Florida, with a 3 s peak gust wind speed of $V_{10\mathrm{m,3s}}$ = 62.6 m/s (140 mi/h). In all cases structural frames spaced at 7.6 m (25 ft) were considered and frame supports were assumed to be pinned. I-shaped frame cross sections with linearly varying web height were considered, and the structural analysis was performed using a linear finite element formulation. The first interior frame (see Fig. 1b) was selected for analysis, and bending moments were evaluated at the knee, the pinch, and the ridge (see

Fig. 1a). For consistency, the pinch in all cases denotes a cross section located 45 % of the distance from the knee to the ridge.

Analysis

The bending moments corresponding to ASCE 7-05 were obtained using the Analytical Procedure (Method 2) for low-rise buildings (ASCE 2006, §6.5) with the design wind speed of $V_{10\,\mathrm{m},3\,\mathrm{s}}$ = 62.6 m/s $(140\,\mathrm{mi/h})$ and open country terrain. The DAD bending moments were then calculated using the *wind*PRESSURE software (Main 2006b). This software uses pressure coefficients that are referenced using the mean wind speed at eave height, with a nominal full-scale averaging time of 1 h. Such hourly averaged wind speeds at eave height can be related to 3 s peak gust wind speeds at 10 m elevation as follows:

$$\frac{V_{H,1\mathrm{h}}}{V_{10\,\mathrm{m},3\mathrm{s}}} = \frac{V_{1\mathrm{h}}}{V_{3\mathrm{s}}}\frac{V_H}{V_{10\,\mathrm{m}}} = \frac{1}{1.52}\left(\frac{H}{10\,\mathrm{m}}\right)^{1/9.5} \tag{1}$$

where the ratio $V_{3\mathrm{s}}/V_{1\mathrm{h}} = 1.52$ was obtained from Fig. C6-4 of ASCE 7-05, and the ratio $V_H / V_{10\mathrm{m}} = (H/10\,\mathrm{m})^{1/9.5}$ follows from a power-law approximation of the mean velocity profile for "open country" terrain (e.g., Simiu and Miyata 2006). Eq. (1) was used to evaluate DAD bending moments corresponding to the 3 s peak gust speed of $V_{10\,\mathrm{m},3\mathrm{s}}$ = $62.6\,\mathrm{m/s}$ $(140\,\mathrm{mi/h})$, for consistency with the bending moments evaluated on the basis of ASCE 7-05. The DAD software utilized the symmetry of the building geometries to calculate bending moments induced by winds from all wind directions, and identified the largest positive and negative moments at the knee, pinch, and ridge (i.e., the maximum and minimum value of each moment over all wind directions).

For both the case of moments based on ASCE-7 plots and the case of moments based on the DAD pressures, no reduction factor to account for wind directionality was used. Had such a directionality factor based on ASCE 7-05 been used, its value would have been $K_d = 0.85$ for both cases. This would not have affected the comparisons presented in this note. The importance factor I and the topographic effect factor K_{zt} were also assumed to be unity for both cases. For the buildings subjected to external DAD pressures both the *observed* and the *estimated* maximum and minimum values of the peaks in the time series of the bending moments were obtained. The estimated peaks correspond to their expected values and were obtained using the procedure described by Sadek and Simiu (2002). Moments induced by internal pressure coefficients from the ASCE 7-05 Standard were obtained for both the "enclosed" and "partially enclosed" cases, which correspond to internal pressure coefficients of $GC_{pi} = \pm 0.18$ and $GC_{pi} = \pm 0.55$, respectively. These moments were added or subtracted from the maximum and minimum moments induced by external pressures in order to produce the worst-case moments. Although measured time histories of internal pressure were available from some tests, these were not used in the present study for the sake of consistency, and even for the bending moments computed using DAD, the contribution of internal pressures was evaluated using the internal pressure coefficients from ASCE 7-05.

Results

Tables 2, 3, and 4, respectively, list the results for the seven buildings due to external pressures only, external pressures with internal pressures for the "enclosed" case, and external pressures with internal pressure for the "partially enclosed" case. The results include maximum and minimum moments at the knee, pinch, and ridge, and the value of

DAD moments divided by ASCE moments. “Est.” and “Obs.” designate the estimated and the observed peaks in the bending moment time series. Uncertainty estimates for the observed peaks shown in Tables 2, 3, and 4 indicate a coefficient of variation of approximately 6 %.

The most interesting results pertain to the knee moments, which have the largest magnitude for all seven buildings and are arguably the most important in design. For building 1 the governing values of the maximum bending moment at the knee based on the ASCE 7-05 plots are larger than the corresponding DAD-based values, the differences being less than 10 %. For all other buildings it is the DAD-based moments that are higher; the differences vary with eave height and roof slope, ranging from about 15 % to 70 % when internal pressure effects are included and from about 20 % to 100 % for external pressures alone. A trend of increasing discrepancy with increasing eave height is evident in the results for buildings 3 through 6. The larger discrepancies for higher eaves are partly accounted for by the fact that the highest eave height considered in the wind tunnel tests of Davenport et al. (1979) was 9.8 m (32 ft), so that the application of these results to higher eave heights in ASCE 7 represents an extrapolation, as pointed out by St. Pierre et al. (2005). At the pinch and the ridge, the DAD-based bending moments consistently exceed those based on ASCE 7-05, in some cases by a factor of four or greater. However, these moments are of much smaller magnitude than the moments at the knee.

In every case, the minimum knee moments produced by DAD pressures exceed the corresponding moments based on ASCE 7-05 pressures. However, in every case these minimum moments are of considerably less magnitude than their positive counterparts.

Efficient member shapes sufficient for the positive moments would also be sufficient for these smaller negative moments.

Concluding Remarks

The maximum knee moments for buildings 2-7 as based on ASCE 7-05 pressures are unconservative, and the degree to which this is the case tends to become more severe as the eave height and the roof slope increase. ASCE 7-05 plots also under-predict bending moments at the pinch and ridge. In most instances the respective bending moments are relatively small and may be absorbed by standard designs imposed by fabrication concerns. Nevertheless, the large discrepancies in these regions indicate that caution should be exercised in optimizing frame cross sections on the basis of the simplified ASCE 7-05 pressures.

In our opinion the principle on which the ASCE 7 pressure plots for low-rise buildings are based is correct, and the integrity of the pioneering work conducted at UWO to develop these plots is unimpeachable. Nevertheless, the technology available at the time of their development forced the code-writers to resort to simplifications mentioned earlier in this note. This unavoidably led to significant errors, as highlighted in some specific examples in this note.

It is pointed out that the conclusions drawn from this study pertain to a restricted sample of buildings with relatively high eaves for open country terrain. Preliminary calculations suggested that discrepancies are smaller for at least some buildings with eave heights lower than those of the buildings examined in this note. For various geometries it may be that the ASCE 7 “pseudo-pressure” coefficients estimate correctly, or over-estimate, some of the wind effects on the frames. To ascertain the extent to which this is the case a more comprehensive set of comparisons is in order and should in our opinion be performed in the future, including comparisons for suburban terrain and for buildings with a wider range of dimensions.

Recent studies (Long 2005, Fritz et al. 2005) showed that wind tunnel measurements on low-rise buildings can depend strongly on the wind tunnel laboratory in which they are conducted. For this reason additional work on how this dependence can be reduced is necessary, so that standard provisions can be developed that are as correct and reliable as the state of the art allows.

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Table 1. Dimensions of buildings selected for analysis (see Fig. 2)

Building number	*H* (m)	*B* (m)	*L* (m)	θ (°)
1	6.1	30.5	61.0	2.4
2	12.2	24.4	38.1	4.8
3	4.9	24.4	38.1	14.0
4	7.3	24.4	38.1	14.0
5	9.8	24.4	38.1	14.0
6	12.2	24.4	38.1	14.0
7	3.7	24.4	38.1	26.6

Table 2. Bending moments due to external pressure alone

Building Number	Peak Type	Knee Moment (kN·m) ASCE-7	Knee DAD Est. (Obs.)	Knee DAD/ASCE-7 Est. (Obs.)	Pinch Moment (kN·m) ASCE-7	Pinch DAD Est. (Obs.)	Pinch DAD/ASCE-7 Est. (Obs.)	Ridge Moment (kN·m) ASCE-7	Ridge DAD Est. (Obs.)	Ridge DAD/ASCE-7 Est. (Obs.)
1	Max	667	615 (611)	**0.92 (0.92)**	62.8	154 (172)	**2.45 (2.73)**	-202	32.0 (37.9)	*-0.16 (-0.19)*
	Min	466	-90.6 (-100)	*-0.19 (-0.22)*	6.8	-103 (-116)	*-15.0 (-17.0)*	-223	-226 (-225)	**1.01 (1.01)**
2	Max	817	976 (924)	**1.19 (1.13)**	215	432 (405)	**2.01 (1.89)**	-108	17.5 (18.1)	*-0.16 (-0.17)*
	Min	1.4	-460 (-485)	*-319 (-336)*	-139	-371 (-361)	*2.67 (2.60)*	-119	-132 (-142)	**1.11 (1.19)**
3	Max	332	448 (398)	**1.35 (1.20)**	37.4	183 (170)	**4.90 (4.55)**	-7.9	15.1 (19.1)	*-1.92 (-2.42)*
	Min	221	-50.1 (-63.8)	*-0.23 (-0.29)*	7.4	-101 (-121)	*-13.7 (-16.5)*	-9.0	-86.0 (-76.6)	**9.50 (8.46)**
4	Max	466	650 (568)	**1.39 (1.22)**	49.7	258 (241)	**5.18 (4.85)**	-51.1	11.6 (16.2)	*-0.23 (-0.32)*
	Min	173	-71.2 (-83.4)	*-0.41 (-0.48)*	-21.3	-176 (-185)	*8.25 (8.69)*	-56.8	-141 (-127)	**2.48 (2.24)**
5	Max	617	856 (785)	**1.39 (1.27)**	108	352 (310)	**3.27 (2.88)**	-85.2	20.5 (22.2)	*-0.24 (-0.26)*
	Min	49.8	-188 (-207)	*-3.77 (-4.15)*	-118	-289 (-273)	**2.44 (2.31)**	-94.6	-186 (-172)	**1.96 (1.82)**
6	Max	800	1190 (1080)	**1.48 (1.35)**	188	476 (459)	**2.53 (2.44)**	-116	50.2 (64.5)	*-0.43 (-0.56)*
	Min	-142	-443 (-467)	*3.11 (3.28)*	-250	-470 (-444)	**1.88 (1.77)**	-129	-244 (-227)	**1.89 (1.76)**
7	Max	218	461 (424)	**2.11 (1.94)**	88.6	331 (304)	**3.74 (3.44)**	43.0	93.0 (86.1)	**2.16 (2.00)**
	Min	4.6	-161 (-158)	*-34.7 (-34.1)*	-132	-319 (-310)	**2.43 (2.36)**	39.5	-26.3 (-26.0)	*-0.67 (-0.66)*

Note: The ratio DAD/ASCE-7 is shown in boldface for the Max or Min with the largest absolute value (i.e., the peak value that would govern in design) and in italics for the Max or Min with the lesser absolute value. If there is a discrepancy between DAD and ASCE 7 as to which peak governs (Max or Min), then the ratios for both peaks are shown in boldface.

Table 3. Bending moments due to external pressure and internal pressure for the “enclosed” case: $GC_{pi} = \pm 0.18$

Building Number	Peak Type	Knee Moment (kN·m)			Pinch Moment (kN·m)			Ridge Moment (kN·m)		
		ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)	ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)	ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)
1	Max	844	792 (788)	**0.94 (0.93)**	74.3	165 (183)	**2.23 (2.47)**	-136	97.8 (104)	*-0.72 (-0.76)*
	Min	288	-268 (-277)	*-0.93 (-0.96)*	-4.6	-114 (-127)	*24.7 (27.6)*	-289	-292 (-291)	**1.01 (1.01)**
2	Max	926	1090 (1030)	**1.17 (1.12)**	231	448 (421)	**1.94 (1.82)**	-86.9	38.2 (38.7)	*-0.44 (-0.45)*
	Min	-108	-570 (-594)	*5.28 (5.51)*	-155	-388 (-377)	*2.50 (2.43)*	-140	-152 (-163)	**1.09 (1.16)**
3	Max	404	520 (469)	**1.29 (1.16)**	41.7	188 (174)	**4.50 (4.18)**	-4.2	18.9 (22.9)	*-4.50 (-5.45)*
	Min	150	-122 (-135)	*-0.81 (-0.91)*	3.1	-105 (-126)	*-34.0 (-40.6)*	-12.8	-89.7 (-80.3)	**7.03 (6.29)**
4	Max	542	726 (644)	**1.34 (1.19)**	52.3	260 (244)	**4.97 (4.65)**	-47.8	14.9 (19.5)	*-0.31 (-0.41)*
	Min	96.6	-147 (-159)	*-1.52 (-1.65)*	-23.9	-178 (-188)	*7.46 (7.85)*	-60.1	-144 (-131)	**2.40 (2.17)**
5	Max	690	929 (858)	**1.35 (1.24)**	108	353 (310)	**3.26 (2.87)**	-82.6	23.2 (24.8)	*-0.28 (-0.30)*
	Min	-23.5	-261 (-280)	*11.1 (11.9)*	-119	-289 (-274)	**2.44 (2.30)**	-97.3	-188 (-175)	**1.94 (1.80)**
6	Max	870	1260 (1150)	**1.44 (1.32)**	189	477 (460)	**2.52 (2.43)**	-110	55.9 (70.2)	*-0.51 (-0.64)*
	Min	-213	-513 (-537)	*2.41 (2.53)*	-251	-471 (-445)	**1.87 (1.77)**	-134	-250 (-233)	**1.86 (1.73)**
7	Max	256	498 (462)	**1.95 (1.81)**	112	355 (328)	**3.17 (2.93)**	74.7	125 (118)	**1.67 (1.58)**
	Min	-32.6	-198 (-195)	*6.08 (5.99)*	-155	-343 (-334)	**2.21 (2.15)**	7.7	-58.1 (-57.8)	*-7.58 (-7.54)*

Note: The ratio DAD/ASCE-7 is shown in boldface for the Max or Min with the largest absolute value (i.e., the peak value that would govern in design) and in italics for the Max or Min with the lesser absolute value. If there is a discrepancy between DAD and ASCE 7 as to which peak governs (Max or Min), then the ratios for both peaks are shown in boldface.

Table 4. Bending moments due to external pressure and internal pressure for the "partially enclosed" case: $GC_{pi} = \pm 0.55$

Building Number	Peak Type	Knee Moment (kN·m)			Pinch Moment (kN·m)			Ridge Moment (kN·m)		
		ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)	ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)	ASCE-7	DAD Est. (Obs.)	DAD/ASCE-7 Est. (Obs.)
1	Max	1210	1160 (1150)	**0.96 (0.95)**	97.8	189 (207)	**1.93 (2.11)**	-1.1	233 (239)	*-207 (-212)*
	Min	-75.7	-632 (-642)	*8.35 (8.48)*	-28.2	-138 (-151)	*4.9 (5.4)*	-425	-427 (-426)	**1.01 (1.00)**
2	Max	1150	1310 (1260)	**1.14 (1.09)**	264	481 (454)	**1.82 (1.72)**	-44.4	80.7 (81.2)	*-1.82 (-1.83)*
	Min	-333	-794 (-819)	*2.39 (2.46)*	-188	-421 (-410)	*2.24 (2.18)*	-182	-195 (-205)	**1.07 (1.13)**
3	Max	551	667 (617)	**1.21 (1.12)**	38.7	185 (171)	**4.77 (4.43)**	3.4	26.5 (30.5)	*7.71 (8.88)*
	Min	2.4	-269 (-283)	*-114 (-119)*	6.1	-102 (-123)	*-16.9 (-20.2)*	-20.4	-97.3 (-87.9)	**4.77 (4.31)**
4	Max	698	882 (800)	**1.26 (1.15)**	57.7	266 (249)	**4.60 (4.31)**	-41.1	21.6 (26.2)	*-0.52 (-0.64)*
	Min	-59.7	-303 (-316)	*5.09 (5.29)*	-29.3	-184 (-193)	*6.27 (6.60)*	-66.8	-151 (-137)	**2.26 (2.06)**
5	Max	841	1080 (1010)	**1.28 (1.20)**	109	353 (311)	**3.25 (2.86)**	-77.1	28.6 (30.3)	*-0.37 (-0.39)*
	Min	-174	-412 (-431)	*2.36 (2.47)*	-119	-290 (-274)	**2.43 (2.30)**	-103	-194 (-180)	**1.89 (1.76)**
6	Max	1010	1400 (1300)	**1.38 (1.28)**	191	479 (462)	**2.51 (2.42)**	-99	67.7 (81.9)	*-0.69 (-0.83)*
	Min	-357	-657 (-681)	*1.84 (1.91)*	-254	-473 (-447)	**1.87 (1.76)**	-146	-261 (-244)	**1.79 (1.67)**
7	Max	332	575 (538)	**1.73 (1.62)**	160	403 (376)	**2.52 (2.35)**	140	190 (183)	**1.36 (1.31)**
	Min	-109	-275 (-272)	*2.52 (2.49)*	-203	-391 (-382)	**1.92 (1.88)**	-57.7	-124 (-123)	*2.14 (2.14)*

Note: The ratio DAD/ASCE-7 is shown in boldface for the Max or Min with the largest absolute value (i.e., the peak value that would govern in design) and in italics for the Max or Min with the lesser absolute value. If there is a discrepancy between DAD and ASCE 7 as to which peak governs (Max or Min), then the ratios for both peaks are shown in boldface.

Figure Captions

Fig. 1. (a) Typical frame showing cross sections selected for analysis; (b) Typical structural system showing frame selected for analysis.

Fig. 2. Gable-roofed building with dimensions indicated.

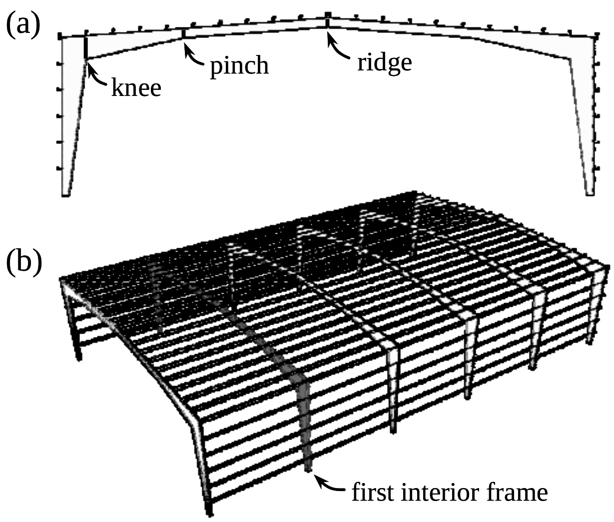

Fig. 1

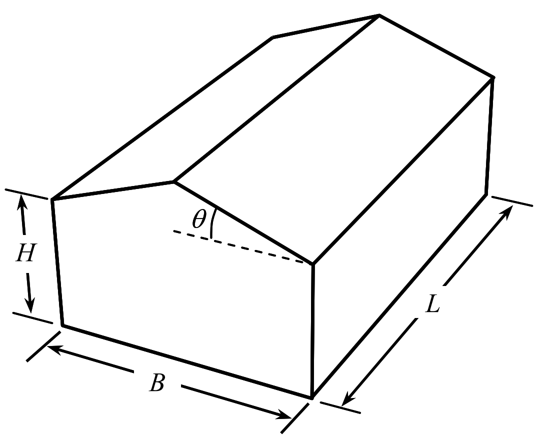

Fig. 2.