Comparison of wind loads calculated by fifteen different codes and standards, for low, medium and high-rise buildings

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ABSTRACT

The paper describes a comparison of wind load calculations on three buildings using fifteen different wind loading codes and standards from the Asia-Pacific Region. The low-rise building is a typical steel portal-framed industrial warehouse building assumed to be located in a rural area. The medium-height building is a 48-metre high office building located in urban terrain. The high-rise building is 183 metres high, also located in urban terrain. The design wind speeds at the top of each building, and other wind properties such as turbulence intensity were prescribed. The comparisons showed varying degrees of agreement. Comments on the differences are given.

INTRODUCTION

Between 2004 and 2007, four International Workshops on Regional Harmonization of Wind Loading and Wind Environmental Specifications in Asia-Pacific Economies were held. A practical outcome of these meetings was a comparison of the wind loads on three typical buildings evaluated by the various wind loading codes and standards across the region. Codes and standards from 15 economies participated in the comparison, including the American Standard ASCE 7-02 [1], and the National Building Code of Canada [2]. The other documents used in the comparison are listed as References [3] to [15]. For some cases, the Eurocode [7] was included in the comparisons.

In an initial comparison, basic wind speeds were specified at the standard meteorological reference position of 10 metres height in flat open terrain. However this resulted in very large differences in building pressures and responses, with much of the differences due to different terrain and height (profile) specifications in the various documents. In the comparisons reported in this paper, design wind speeds *at the heights of the tops of the buildings* in the chosen terrain were specified. Wind speeds with averaging times of 3-seconds, 10-minutes and 1-hour were all specified, and participants selected the appropriate ones according to the averaging time used in their own code or standard. For the medium- and high-rise buildings, first mode natural frequencies and critical damping ratios were also specified.

This paper presents the main results of the comparisons and discusses some reasons for the differences. The paper is a shorter version of one presented at the Fourth International Conference on Advances in Wind and Structures (AWAS 08), held in Jeju, Korea, May 29-31, 2008. It is presented here for the interest of a wider audience on the eastern side of the Pacific.

LOW-RISE BUILDING

The low-rise building is a typical steel portal-framed industrial warehouse building assumed to be located in a rural area (Figure 1). Participants were asked to calculate wind loads for the structural design of the portal frames at the end of the building, a large roller door (3m x 4m) on one wall, and a small window ($1m^2$) on the opposite wall. Internal pressures from a large opening were included for some wind directions. Design wind speeds at 6m height of 39 m/s, 26 m/s and 23 m/s were specified for the averaging times of 3-seconds, 10-minutes, and 1 hour, respectively. Open terrain all around was specified.

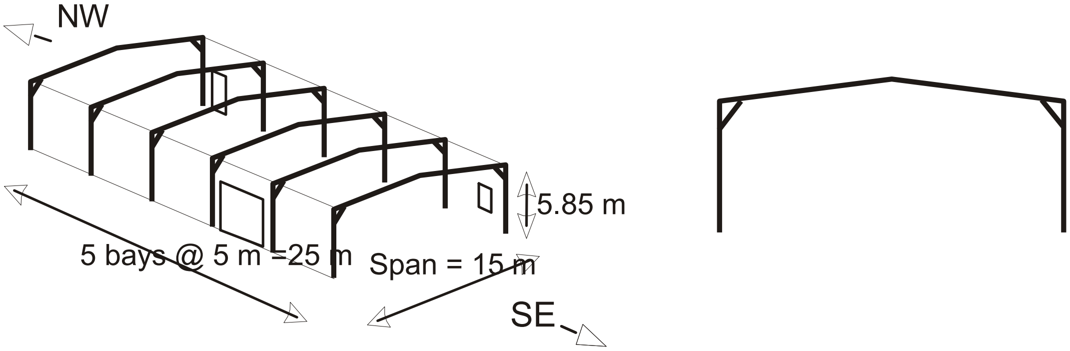


Figure 1: Low-rise building (Building 1).

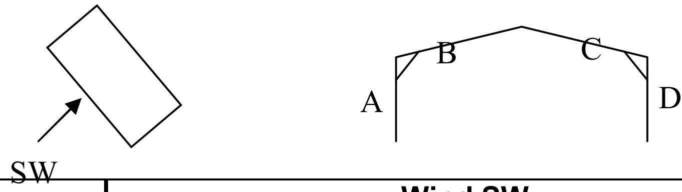
Table 1 gives a comparison of net pressure coefficients across the four faces (A → D) of a typical end frame, for SW wind with the large opening (roller door) being considered. The coefficients of variation range between 20 and 31%.

For the case of a wind parallel to the ridge line, considering the building to be closed ($Cp_i = 0$), the coefficients of variation for the pressure coefficients ranged from 44 to 61%.

Table 2 compares the maximum design pressures (or suctions) on the roller door (SW wall) and the small window (NE wall). The comparison here is better, the coefficient of variation being within 13 and 26%.

Since almost all the parameters have been normalized in this example, the only variable is the coefficient of pressure, and the variation does appear to be rather large. The only explanation would seem to be that different standards have sourced different wind tunnel test results on which the coefficients have been based.

Table 1: Net Pressure Coefficients Across Gable Walls & Roof; Wind-SW, C_{pi} =+0.7 (Large opening in SW Wall)



Country	Wind SW			
	A	B	C	D
Australia/New Zealand*	-0.14	-1.33	-1.01	-1.07
Canada$^+$	0.45	-2.70	-1.70	-1.50
Euro	0.02	-2.40 → -1.30	-1.30	-1.05
India	0.0	-1.61	-1.11	-0.96
Indonesia	-0.14	-1.30 → -0.93	-0.93 → -0.88	-1.07
Japan	-0.10	-1.49	-1.49 → -1.17	-1.14
Malaysia	-0.15	-1.23	-0.83	-1.00
Philippines	0.50	-1.55	-1.05	-0.95
Taiwan	0.07	-0.76	-0.76	-0.63
United States	-0.09	-1.77	-1.23	-1.13
Vietnam	0.12	-1.38	-1.36	-1.42
Mean	***0.05***	***-1.53***	***-1.14***	***-1.08***
Coefft. of Variation %	***-***	***31***	***23***	***20***

* shape factors (including area reduction and combination factors)

$^+$ including gust factors

Table 2: Maximum wind pressures (+ pressure / - suction) on the 3m x 4m roller door and the 1m^2 window, kPa

Country	Roller Door (SW Wall)	Window (NE Wall)
Australia/NZS	0.64, -0.46	0.80, -1.19
Canada	0.57, -0.59	0.68, -0.68
China	0.64, -0.80	0.64, -1.43
Euro	0.81, -0.90	1.13, -1.24
Hong Kong	0.89, ------	0.89, -1.25
India	0.71, -0.51	0.71, -0.96
Indonesia	0.64, -0.46	0.80, -1.19
Japan	0.62, -0.75	0.62, -1.51
Korea	0.96, -0.83	0.96, -1.08
Malaysia	0.66, -0.47	0.82, -1.22
Philippines	0.75, -0.84	0.84, -1.73
Taiwan	0.61, -0.65	0.76, -1.37
Vietnam	0.91, -0.53	0.91, -1.33
United States	0.55, -0.62	0.67, -0.93
Mean	***0.71, -0.65***	***0.80, -1.22***
Coefft.. of Variation %	***13, 16***	***14, 26***

MEDIUM-RISE BUILDING

Building 2 has horizontal dimensions of 60 m by 30 m with a roof height of 48 m. The building is assumed to be of reinforced concrete construction, with a façade consisting of mullions spaced at 1.5 metres (Figure 2).

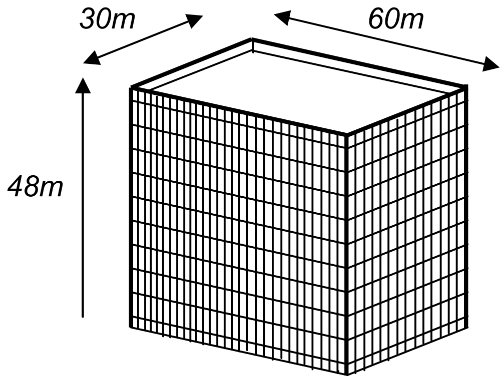


Figure 2: Medium-rise building (Building 2).

The building is assumed to be air-conditioned with non-opening windows, and can be assumed to be effectively sealed with regard to internal pressures. The along-wind base bending moment and shearing force were required to be calculated for wind directions normal to the 60 m wall. Cladding pressures on window elements near the corners at the top level were also calculated. The 3-second, 10-minute and 1-hour wind speeds at the top of the building were specified as 56 m/s, 36 m/s and 33 m/s respectively, and a turbulence intensity of 0.200 at the top of the building was assumed. The resonant response for this building was required to be considered by some codes and standards, and, for this purpose, the first-mode natural frequency of 1.2 Hz, and critical damping ratio of 2% were specified.

The calculated values for along-wind base shears Q and base bending moments M are shown in Table 3 and compared in Figure 3. It may be a matter of course that Q and M show a clear correlation. Indonesia [13] shows the highest values, (7,477kN and 210MNm), and China [8] shows the lowest combination, (3,282 kN and 99MNm). The Indonesian values are more than double the Chinese values. The coefficient of variation is estimated at 22% for both the base shear and the base bending moment. Considering the given harmonized condition specifying the same design wind speed at the top, the coefficient of variation, 22%, is larger than expected. Singapore (draft standard), Vietnam [15], Australia/New Zealand [12], Malaysia [9], and Indonesia compose a higher magnitude group (see Circle A in Fig.3). Japan [3], Korea [11[and Canada [2] (Circle B), India [6] and Hong Kong [5] (Circle A′) and the Philippines [4] compose a medium magnitude group. Thailand [10] and Taiwan [14] (Circle C′), the U.S. [1] and China [8] compose a lower magnitude group. The US and the Philippines are in Circle C. These groups closely correspond to several groups related to their origins, as mentioned in the next section for Building C.

Table 4 shows the cladding pressures on window elements near the corners at the top level. The coefficients of variation for positive cladding pressures and negative cladding pressures are estimated at 22% and 23%, and are similar to those for along-wind base shears and base bending moments.

Table 3: Along-wind base shears and bending moments for Building 2

Country/Region		Code/Standard	Base Shear *Q* (kN)	Base Bending Moment *M* (MN.m)
Australia/New Zealand	AN	AS/NZS1170.2: 2002	5,727	150
Canada	NB	NBCC (2005)	5,332	142
China	CH	GB50009-2001	3,282	99
Hong Kong	HK	CP-2004	4,573	116
India	IN	IS875(Part 3)-1987	4,957	131
Indonesia	IA	SNI-03-1727	7,477	210
Japan	JA	AIJ-RLB-2004	5,061	132
Korea	KO	KBC (2005)	5,534	134
Malaysia	MA	MS1553-2002	5,698	152
Philippines	PH	NSCP-2001	5,026	128
Singapore	SI	(draft)	6,556	163
Taiwan	TA	TBC	3,738	100
Thailand	TH	EIT-1018-46	3,737	97
United States	US	ASCE 7-05	4,108	117
Vietnam	VI	TCVN2737-1995	6,423	165
Mean			***5,149***	***136***
***Coefficient of Variation* (%)**			***22***	***22***
Eurocode	EU		6,042	182

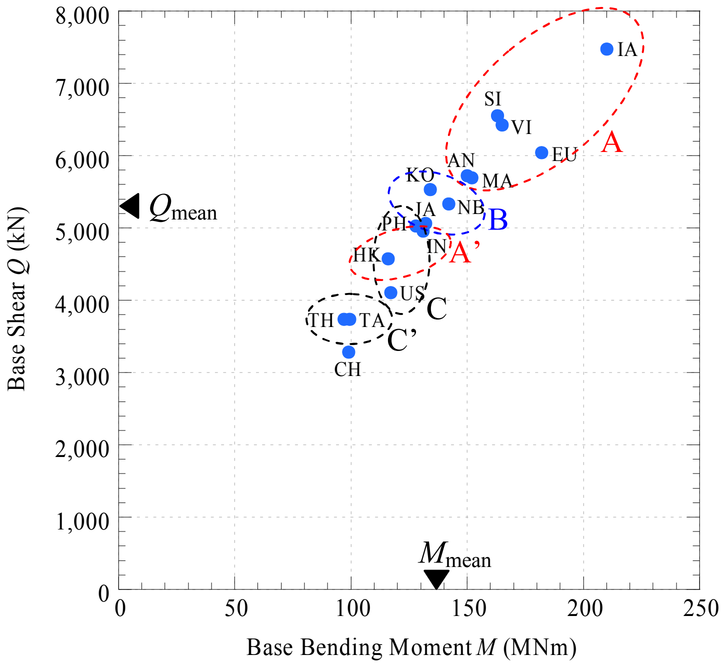


Figure 3: Relation between base shear and base bending moment for the medium-rise building (Building 2).

The Vietnam Norm [15] shows the highest positive cladding pressure, 2.44kPa, but the highest negative (i.e. lowest magnitude) pressure, −1.83kPa. China shows the lowest positive cladding pressure, 1.22kPa, and a relatively high negative pressure, −2.44kPa, i.e. a lax provision. On the other hand, Australia/New Zealand, Malaysia, Indonesia and Singapore (Circle A) as a group derived from the same source, show the most unfavorable combination of positive and negative pressures, such as (2.3kPa and −3.8kPa).

Table 4: Cladding pressures for Building 2

Country/Region		Code/Standard	Positive Cladding Pressure $P+$ (kPa)	Negative Cladding Pressure $P-$ (kPa)
Australia/New Zealand	AN	AS/NZS1170.2:2002	2.25	–3.67
Canada	NB	NBCC (2005)	1.80	–2.11
China	CH	GB5009-2001	1.22	–2.44
Hong Kong	HK	CP-2004	1.87	–2.62
India	IN	IS875(Part 3)-1987	1.55	–2.26
Indonesia	IA	SNI-03-1727	2.24	–3.64
Japan	JA	AIJ-RLB-2004	2.14	–2.37
Korea	KO	KBC (2005)	1.53	–2.54
Malaysia	MA	MS1553-2002	2.26	–3.70
Philippines	PH	NSCP-2001	1.32	–2.85
Singapore	SI	(draft)	2.26	–3.67
Taiwan	TA	TBC	1.58	–2.95
Thailand	TH	EIT-1018-46	1.86	–2.23
United States	US	ASCE 7-05	1.41	–2.56
Vietnam	VI	TCVN2737-1995	2.44	–1.83
Mean			***1.85***	***–2.76***
***Coefficient of Variation* (%)**			***22***	***23***
Eurocode	EU		1.69	–2.47

HIGH-RISE BUILDING

The high-rise building was 183 metres high, with horizontal dimensions of 46 m and 30 m located in urban terrain (Figure 4). This building was previously used as a benchmark test building for aeroelastic wind-tunnel tests, known as the CAARC Building [16]. The building was assumed to have an average density of 160 Kg/m^3, and natural frequencies in both sway directions of 0.20 Hertz. The sway mode shapes were assumed to be linear. The structural damping, as a fraction of critical, was specified to be 0.012 for ultimate limit states (base shear and bending moment), and 0.008 for serviceability limits states (accelerations at the top of the building). For wind directions normal to the 46 m wall, base bending moments and shears, and peak accelerations at the top of the building, were required to be calculated. Both along-wind and cross-wind responses were calculated, when the particular codes and standards allowed these calculations to be made. However, not all codes and standards allowed cross-wind response and accelerations to be calculated.

Design wind speeds for three different averaging times were specified to cover the range of times adopted by various codes and standards in the region. Values were given for both ultimate limit states (base shear and bending moment) and for serviceability limits states (accelerations at the top of the building). The values of design wind speeds are tabulated in Table 5.

Table 5: Design wind speeds (m/s) for Building 3 (183 m)

Averaging time	Ultimate limit states	Serviceability limit states
3-seconds	59	35
10-minutes	41	25
1-hour	37	22

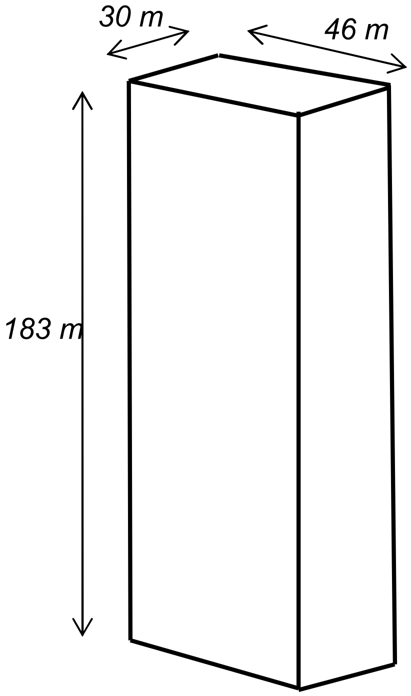


Figure 4: High-rise building (Building 3).

The building has a significant amount of resonant dynamic response to wind which complicated the evaluation of base shear, bending moments and acceleration at the top of the building.

The calculated values for along-wind shear and bending moments in Table 6 can be grouped into two main groups – a higher magnitude group including the AIJ Recommendations [3], and Australia/New Zealand [12] and the closely related Malaysia [9] and Indonesian Standards [13]. A lower group includes ASCE 7 [1], Taiwan [14] and the Philippines [4]. China [8] has one of the lowest values – this can be explained, at least partially by the fact that it does not include a 'background' component in its calculation method. The coefficients of variation of 14 to 15% are reasonable values considering the complexities of the calculations. Much of the differences in the calculated along-wind response can be traced to differences in the velocity profiles over the height of the building.

Table 6: Along-wind base shears and bending moments for Building 3

Country/region	Code/Standard	Base shear (kN)	Base bending moment (MN.m)
Australia/ New Zealand	AS/NZS1170.2 :2002	21500	2085
Canada	NBCC (2005)	19844	1994
China	GB5009-2001	13867	1554
Hong Kong	CP-2004	15817	1583
India	IS875(Part 3)-1987	17648	1819
Indonesia	SNI-03-1727	21292	2264
Japan	AIJ-RLB-2004	21540	2162
Korea	KBC (2005)	19637	2017
Malaysia	MS1553-2002	21870	2110
Philippines	NSCP-2001	16100	1574
Taiwan	TBC	17076	1748
Thailand	EIT-1018-46	15091	1539
United States	ASCE 7-05	18305	1795
	Mean	*18430*	*1860*
	Coefft. of Variation(%)	*14.8*	*14.0*

Only six documents include a calculation of cross-wind shear and base moment, and, in fact, that assigned to ASCE 7 [1] is actually from the web site of the Natural Hazards Group at the University of Notre Dame (www.nd.edu/~nathaz/database). The comparisons for *cross*-wind shear and bending moment in Table 7 show slightly greater variability than the along-wind responses, but the coefficients of variation (16-17 %) are quite small considering the uncertainty in the phenomenon (random vortex shedding) driving the cross-wind response. For example, there is considerable variability in the spectral densities of the cross-wind forces for buildings of some cross sections (Holmes and Flay, [17]). Also there are other differences in the calculation methods – for example the Australia/New Zealand Standard [12] neglects any background contribution to the cross-wind response, whereas this is included in the AIJ Recommendations [3] and the ASCE 7/ U.N.D. calculations.

Nine codes and standards permitted calculation of accelerations at the building top. The National Building Code of Canada [2] allows calculation of only cross-wind accelerations, and the Indian Standard [6] considers only along-wind response. The variability in the calculated accelerations however was reasonable (coefficients of variation of 17-18%) considering the number of variables involved in the calculations, and all codes and standards agree that the cross-wind acceleration is greater than the along-wind acceleration, for the specified wind direction.

Table 7: Cross-wind base shears and bending moments for Building 3

Country/region	Code/Standard	Base shear (kN)	Base bending moment (MN.m)
Australia/ New Zealand	AS/NZS1170.2 :2002	11200	1365
Indonesia	SNI-03-1727	9511	1191
Japan	AIJ-RLB-2004	14540	1779
Malaysia	MS1553-2002	11200	1365
Taiwan	TBC	12006	1477
Thailand	EIT-1018-46	12272	1497
United States	ASCE 7-05	15360	1893
	Mean	***12300***	***1510***
	Coefft. of Variation(%)	***16.5***	***16.6***

Perhaps surprisingly, the coefficients of variation for both along-wind and cross-wind responses for this building were relatively small compared with those for the other two buildings – in the range of 14 to 18%. This may be because many of the methods were inter-related. For example, several documents use variations of the methods used in the American [1] and Australian/New Zealand Standards [12] or National Building Code of Canada [2] for along-wind response, or from the AIJ (Japan) Recommendations [3] for cross-wind response.

DISCUSSION AND CONCLUSIONS

1. The coefficient of variation for the results for the low-rise building (Building 1) is somewhat large, considering its comparative simplicity as opposed to the complexities in the Buildings 2 and 3.

2. For the medium-rise building (Building 2), no significant correlation was observed between the along-wind load effects, i.e. base shears and base bending moments, and dynamic response factors or gust loading factors. However, some correlation was observed between cladding pressures and net peak cladding force coefficients. It was also clearly recognized that some clusters show almost the same or similar behaviors because of the existence of some common source codes/standards. The mean values and coefficients of variation of the fifteen codes/standards in the Asia-Pacific region were calculated, and the coefficients of variation were estimated at around 22% - 23% for both along-wind overall load effects and cladding pressures. This relatively high coefficient of variation is a little surprising, because the calculation was made under the well harmonized condition, where the design wind speed and the turbulence intensity at the top of the building, and the first mode damping ratio and natural frequency are all given. It should be noted that the variation would become more significant if those values were not specified, e.g. only giving the basic wind speed at 10m height. It should also be noted that the estimated statistical values such as mean value or coefficient of variation have only limited meaning because of the inter-relation of the codes/standards.

3. The high-rise building (Building 3) has a significant amount of resonant dynamic response to wind which complicates the evaluation of base shear, bending moments and acceleration at the top of the building. Not all codes and standards in the Asia-Pacific region allowed for cross-wind response and accelerations to be calculated. Perhaps surprisingly, the coefficients of variation for both along-wind and cross-wind responses were relatively small – in the range of 14

to 18%. This may be because many of the methods were inter-related; for example, several documents use variations of the methods used in the American and Australian Standards and National Building Code of Canada for along-wind response.

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REFERENCES

[1] A.S.C.E. Minimum design loads for buildings and other structures. ASCE Standard ASCE/SEI 7-05. American Society of Civil Engineers, Reston, Virginia, 2006.

[2] National Research Council of Canada, (2005). National Building Code of Canada. NRCC Ottawa.

[3] Architectural Institute of Japan., Recommendations for loads on buildings. AIJ-RLB-2004, Tokyo.

[4] Association of Structural Engineers of the Philippines, National structural code of the Philippines, 5th edition, NSCP-2001.

[5] Buildings Department, Hong Kong Special Administrative Region, China, Code of practice on wind effects – Hong Kong, 2004.

[6] Bureau of Indian Standards, Indian Standard Code of Practice for design loads (other than earthquake) for buildings and structures. Part 3 – Wind loads, IS: 875 (Part 3) -1987.

[7] C.E.N. (European Committee for Standardization), Eurocode 1: Actions on structures - Part 1-4: General actions - Wind actions, prEN 1991-1-4.6, C.E.N., Brussels, 2004.

[8] China Architecture and Building Press, Load code for the design of building structures, China National Standard, GB 50009-2001 (revised), 2006.

[9] Department of Standards, Malaysia, Code of practice on wind loading for building structure, Malaysian Standard, MS 1553: 2002.

[10] Engineering Institute of Thailand, Wind loading code for building design, EIT Standard 1018-46, 2003.

[11] Korean Government Guidelines of Korean Building Code – Structural (2005). KGG-KBCS-05.

[12] Standards Australia/Standards New Zealand, Structural design actions. Part 2 Wind actions, AS/NZS1170.2:2002.

[13] Standard Nasional Indonesia, SNI 03-1727.

[14] Taiwan Architecture and Building Research Institute (2006), Specifications for building wind-resistant design, (Wind load provisions of Taiwan Building Code).

[15] Tieu Chuan Viet Nam (1995), Loads and Actions Norm for Design, TCVN 2737 – 1995.

[16] R.L Wardlaw and G.F. Moss. Thom, A standard tall building model for the comparisons of simulated natural winds in wind tunnels, Proceedings, 3rd International Conference on Wind Effects on Buildings and Structures (Tokyo, Japan, 1971) pp 1245-1250, Saikan Shuppan, 1971.

[17] J.D. Holmes and R.G.J. Flay, Cross-wind force spectra. Journal of Wind Eng. (ISWE), 4(2007), 13-18.