International high-frequency base balance benchmark study

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ABSTRACT

A summary of the main results from an international comparative study for the high-frequency base balance is given. Two buildings were specified – a 'basic' and an 'advanced' building. The latter had more complex dynamic response with coupled modes of vibration. The predicted base moments generally showed good agreement amongst the participating groups, but less good agreement was found for the roof accelerations which are dominated by the resonant response, and subject to measurement errors for the generalized force spectra, and to varying mode shape correction techniques, and different methods used for combining acceleration components.

1. INTRODUCTION

The High Frequency Base Balance (HFBB) (also known as the High-Frequency *Force* Balance (HFFB)), has been widely used as a wind-tunnel tool for determining overall wind loading and dynamic response of tall buildings to wind, for about thirty years. The basic principles have been described by Tschanz and Davenport (1983) and Boggs and Peterka (1989).

However, although the same basic principles are applied, a variety of approaches have been used to deal with, for example:

- Calculation of resonant response
- Twist (torsional) modes of vibration
- Nonlinear mode shapes
- Coupled mode shapes
- Closely spaced frequencies
- Combined acceleration components

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At a meeting held at the 12th International Conference on Wind Engineering (Cairns, Australia, July 2007), it was agreed that an International HFBB Comparison project should be initiated (Holmes *et al.*, 2008). Two model tall buildings were defined, and various participating wind-tunnel laboratories manufactured their own models and carried out the tests, and subsequently presented results for comparison. Groups will not be named explicitly in any reporting of the study.

The two buildings comprised:

- A ‘basic’ test building intended primarily for as a benchmark for newer groups (‘Building B’) (Figure 1),
- An ‘advanced’ building specification for more experienced groups (‘Building A’) (Figure 2).

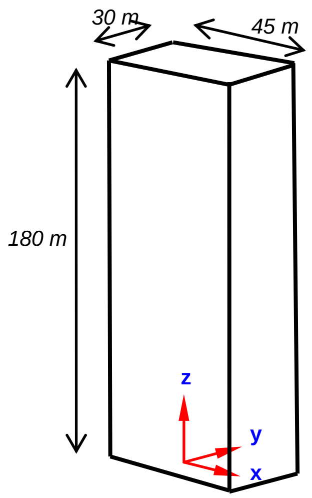

Fig. 1 ‘Basic’ Building B for HFBB Comparison

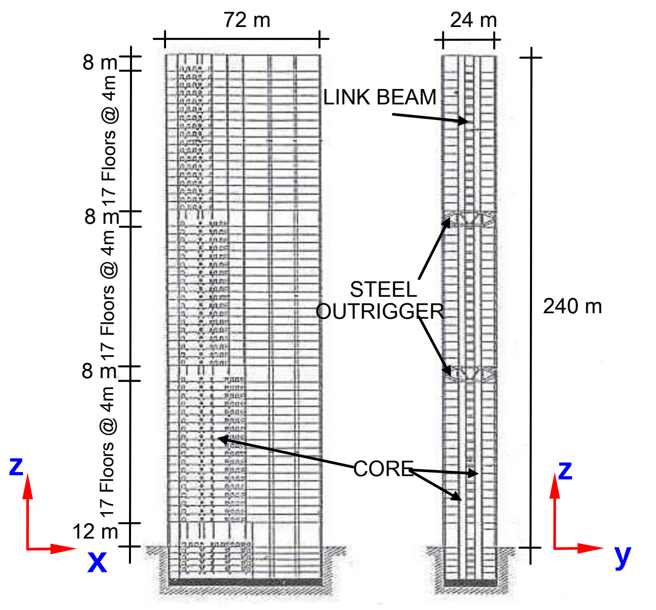

Fig. 2 ‘Advanced’ Building A for HFBB Comparison

The experimental part of this study is now complete – eight groups have submitted results. The eight groups include four universities and four commercial wind testing companies. Geographically the groups were from Canada (two), and one from each of United States, Australia, Korea, China, Japan and Hong Kong. There were no participants from Europe. The study has been endorsed by the International Association for Wind Engineering (www.iawe.org) and the full specifications for the study can be found on the website of the IAWE.

The models used by one of the participating groups are shown in Figure 3.

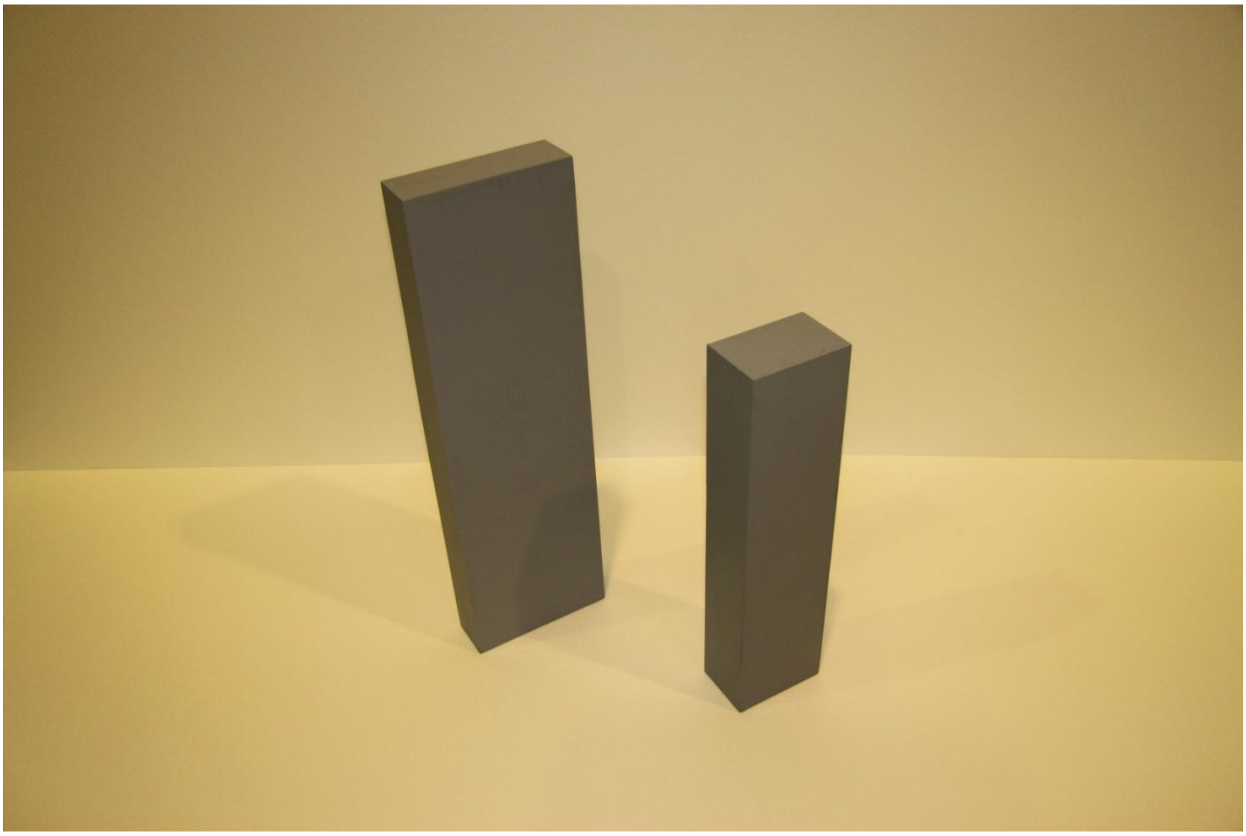

Fig. 3 The two building models used by one of the participating groups
(Building A on the left; Building B on the right)

This paper presents some results from the analysis of the data for base moments (both bending and torsional moments), and for accelerations at the top of the buildings.

2. BASIC BUILDING COMPARISON

The 'basic' building is 180 metres in height and has cross sectional dimensions of 45 metres by 30 metres (Figure 1). The sway frequencies were prescribed to be 0.20 and 0.23 Hertz, and the third (twist) mode has a frequency of 0.40 Hertz. All three mode shapes are assumed to vary linearly with height. The characteristics of the approaching boundary layer flow (applicable to both the basic and advanced building) are listed in Appendix A.

Seven groups submitted results for this building. One group of the total of eight did not submit results for the basic building.

2.1 Base moment comparison - Building B

Figure 4 shows the mean base moments for a mean wind speed at the top of the building of 40 m/s, from the seven different laboratories. The mean sway moments show good agreement, with coefficients of variation of about 8-10% for the largest values. The mean torsional moments show more scatter, although the magnitudes are considerably smaller than the sway moments. To account for varying air density used by the various groups, the results have been corrected to a sea level density of 1.20 Kg/m^3. These corrections varied between 0 and 8%.

The mean base moments have been averaged over the seven sets of results, and converted to coefficients by the following:

$$C_M = \frac{M}{0.5\rho \bar{U}_h{}^2 b h^2} \tag{1}$$

where $\bar{U}_h$ is the mean wind speed at the top of the building (40 m/s in this case), b is the breadth (45 metres), and h is the building height (180 metres). A sea level air density of 1.2 Kg/m^3 has been assumed for the calculation of coefficients.

The resulting coefficients have been plotted against wind direction, and are shown in Figure 5. In this plot, the mean torsional moment, $\bar{C}_{Mz}$ has been multiplied by 10 for more clarity. The mean sway moment coefficients, $\bar{C}_{Mx}$ and $\bar{C}_{My}$ can be directly compared with those obtained from an earlier benchmark study for an aeroelastic model – the 'CAARC' building, as reported by Melbourne (1980). Generally the agreement is good, although the individual measurements in the earlier study showed more scatter, perhaps reflecting less reliable experimental techniques at that time.

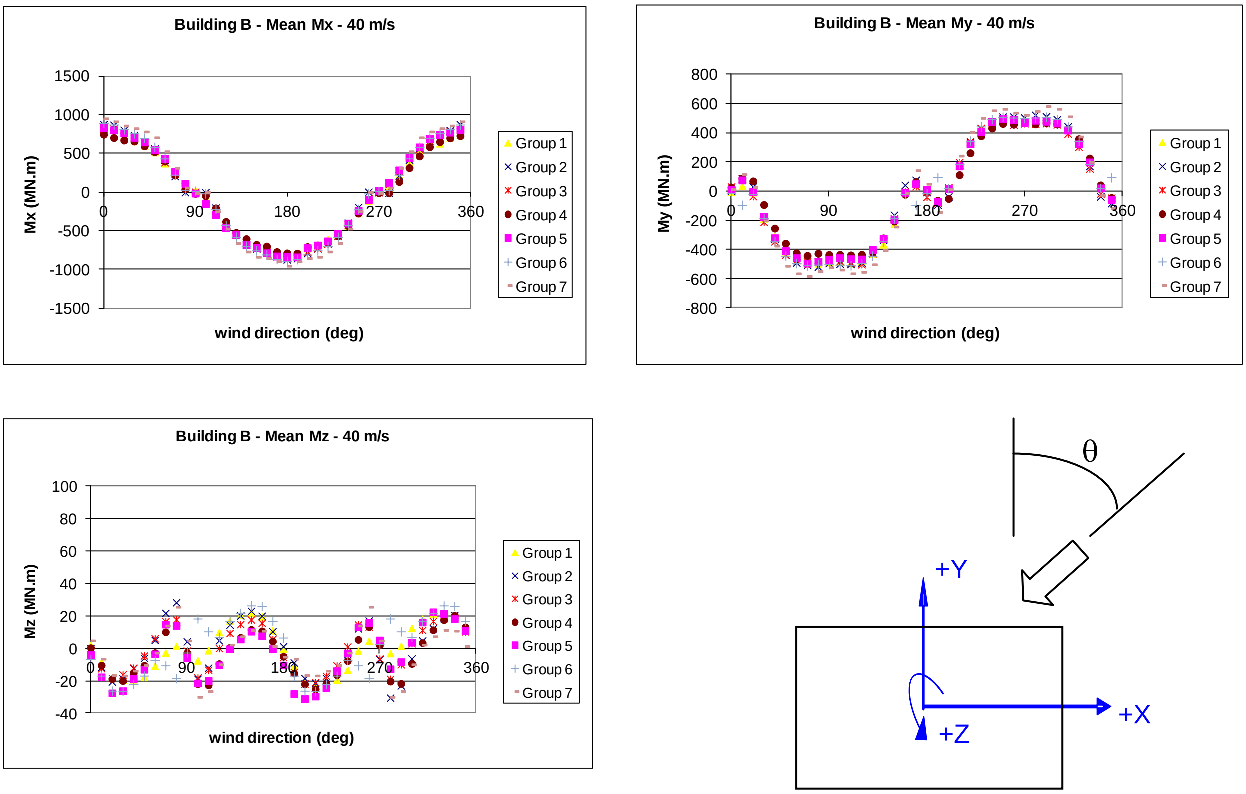

Fig. 4 Mean base moments ($\bar{U}_h$ = 40 m/s) and sign convention (Building B)

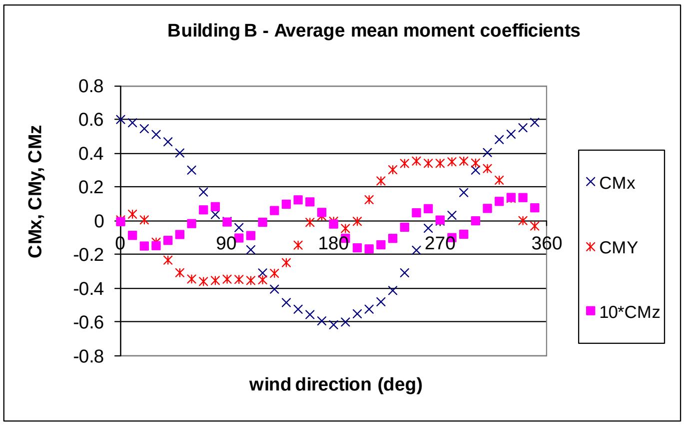

Fig. 5 Averaged mean moment coefficients for Building B

The predicted *maximum* base moments about the x axis for a wind speed of 40 m/s and critical damping ratio of 2.5%, are shown in Figure 6. *Minimum* base moments about the y-axis (note the sign convention in Figure 1) are shown for the same wind speed and damping in Figure 7. The maximum moments about the z axis (i.e. twisting moments) are shown in Figure 8.

The maximum base moments include fluctuating resonant components obtained by computation. These are sensitive to the measurements of spectra by the HFBB, the method of integration and the mode shape correction in the case of the torsional moment (M_z). Note that the sway moments should not require mode shape corrections for Building B, as they were specified to be linear for this building.

Probably as a result of the above complexities and sources of differences, the coefficients of variation of these results are greater than those for the mean base moments – about 15-20%, but are generally acceptable.

2.2 Comparison of accelerations – Building B

The participating groups were also asked to provide resultant accelerations at the top of the building at the geometric centre, and at a corner of the roof, for three different wind speeds. The corner accelerations incorporate the twist component of motion, as well as the sway motions. Results were provided for two damping ratios – 1% and 2.5%, although in practical terms the former value is the more relevant one for serviceability assessments

Figure 9 shows predicted maximum corner accelerations for the building with damping of 1% of critical, and for a wind speed at the top of the building of 20 m/s. Note that two groups did not provide data for these conditions.

There is a fair amount of scatter in the predicted maximum accelerations (for any wind direction), ranging between 7.2 and 13.3 millig s. The latter value would put the building into the 'unacceptable' category for a 1-year return period according to some criteria for occupant comfort (see for example: Holmes, Kwok and Ginger, 2012, Figure 5.9).

Acceleration calculations from a high-frequency base balance are dominated by the resonant components of the response, which are not measured but are calculated from the spectral densities of the measured base moment spectra. They are thus sensitive to measurements of the spectra at high frequencies, particularly at the high reduced frequencies associated with serviceability wind speeds. Factors such as the methods used for windowing and smoothing of the spectra, as well as mode shape corrections to both sway and twist modes, and the method used to integrate for the resonant response, also play a role.

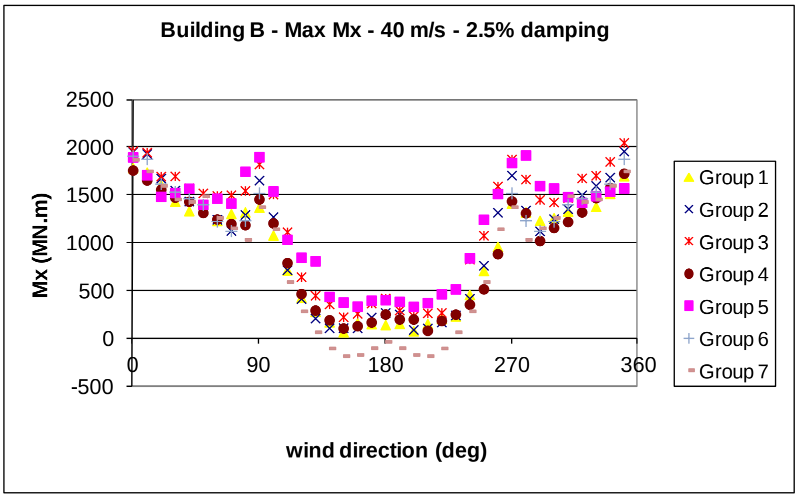

Fig. 6 Predicted maximum sway moments about the x-axis

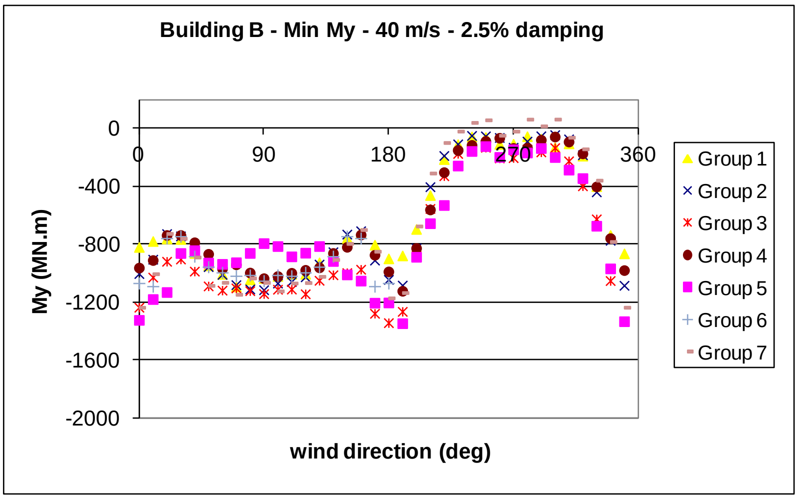

Fig. 7 Predicted minimum sway moments about the y-axis

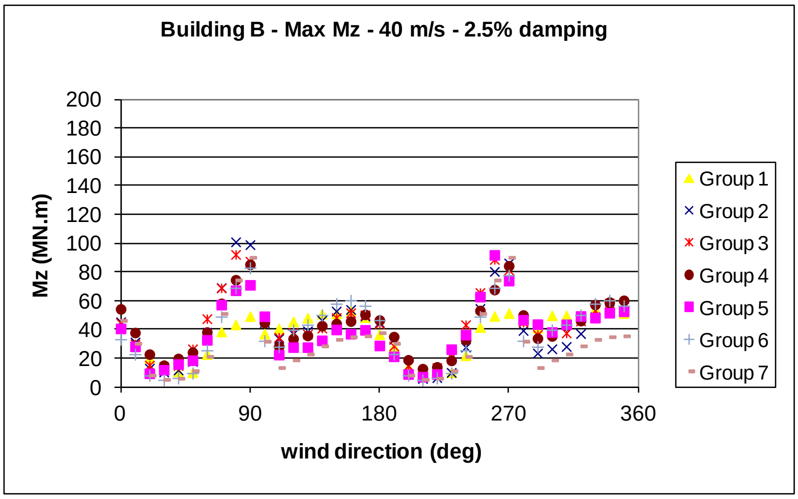

Fig. 8 Predicted maximum twist moments about the x-axis

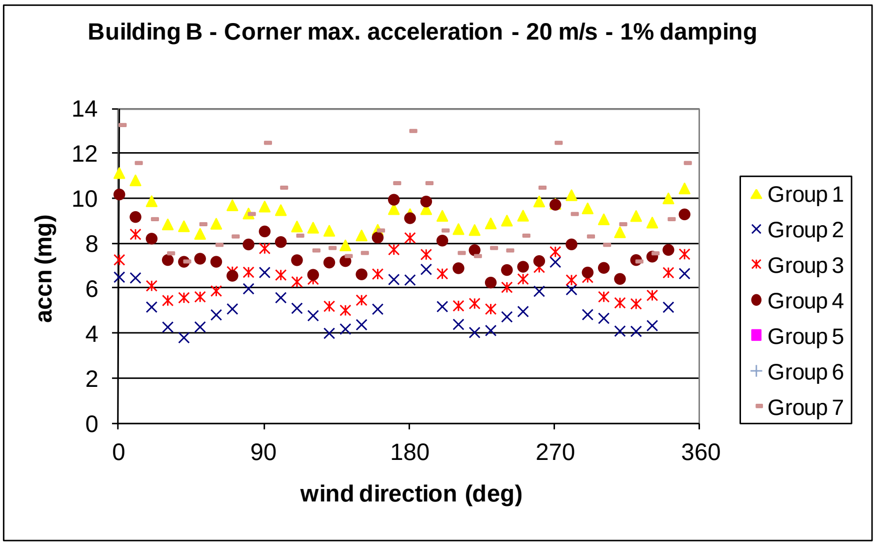

Fig. 9 Predicted corner accelerations for a mean wind speed of 20 m/s, 1% damping

Figure 10 shows similar results to those in Figure 9, but for a mean wind speed at the top of the building of *30 m/s*. This wind speed represents approximately a 5 to 20 year return period value for many locations in the world. The measured peak accelerations are approximately three times those in Figure 9, corresponding to a variation with mean wind speed with an average exponent of about 2.7. The scatter between groups is generally lower than that in Figure 9, probably related to the greater measurement accuracy in measuring spectra at the lower reduced frequencies associated with the higher wind speed.

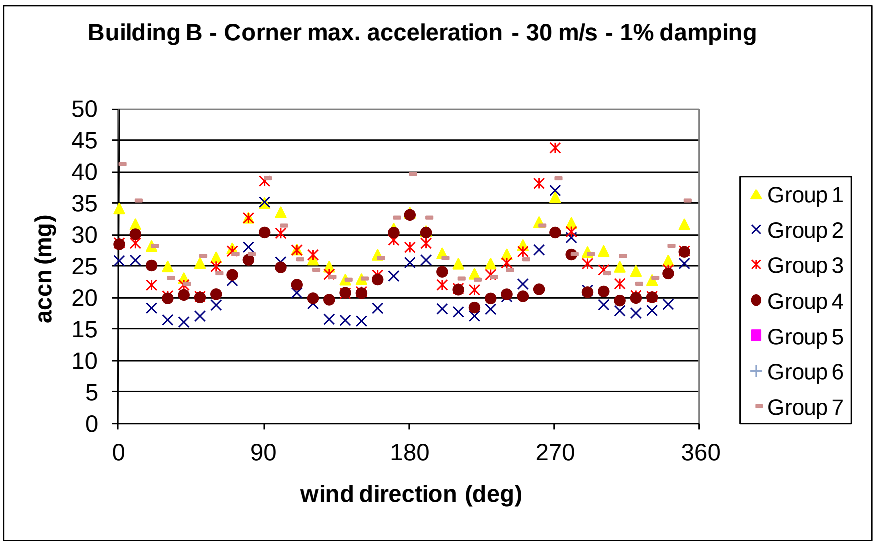

Fig. 10 Predicted corner accelerations for a mean wind speed of 30 m/s, 1% damping

3. ADVANCED BUILDING COMPARISON

The ‘advanced’ building, denoted as Building A, is a modified version of a 60-story benchmark building with height of 240 metres, already in use for vibration control applications (Tse *et al*., 2007). This building is also of uniform rectangular cross-section with dimensions of 72 metres by 24 metres. The dynamic properties are summarized in Table 1 and Figure 11. Only the first three of the original six modes have been specified for the HFBB benchmark tests. The main modification is the introduction of more lateral-torsional coupling for the second and third modes. Hence, the three modes are all coupled to provide more of a challenge for the HFBB processing techniques.

Table 1. Dynamic Properties of Building A

Mode	Coupling of Mode Shape	Frequency (Hz)	Damping (% of critical)
1	sway-y & twist	0.231	1 and 2.5
2	sway-x & twist	0.429	1 and 2.5
3	twist, sway-x & sway-y	0.536	1 and 2.5

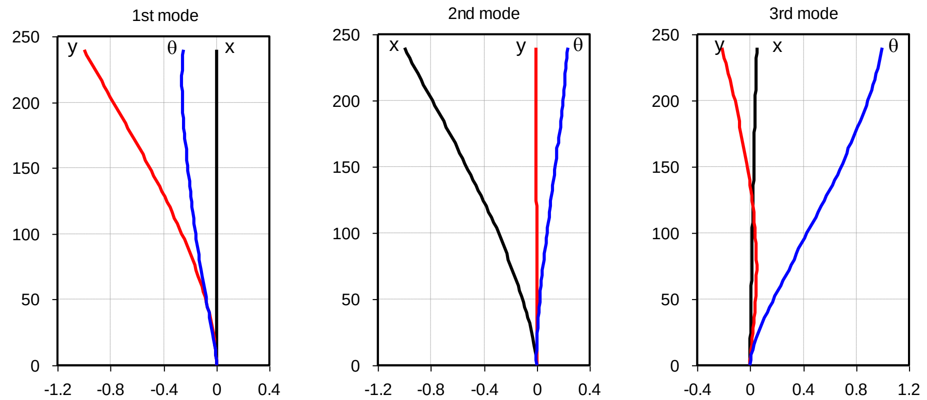

Fig. 11 Mode shapes for Building A

3.1 Base moment comparison – Building A

Eight groups submitted results for this building, which have been corrected to a sea level density of 1.20 Kg/m^3 prior to the following comparisons. Similar to the Building B, the mean sway moments show reasonably good agreement while the mean torsional moments show slightly more scatter. The coefficients of variation are approximately 20%. To avoid redundancy, only the results for a mean wind speed at the top of the building of 30 m/s are showed in Figure 12.

The dynamic base moments were subsequently calculated by each group based on the supplied structural dynamic properties, including natural frequencies, mode shapes, mass distribution and story mass centres. The predicted *maximum* base moments about the x axis for a wind speed of 30 m/s and critical damping ratio of 1.0%, are shown in Figure 13. *Minimum* base moments about the y-axis are shown for the same wind speed and damping in Figure 14. The maximum torsional moments about the z axis are shown in Figure 15. Higher coefficients of variation of over 20-30% are evidently observed, in particular for the torsional moments, which are largely attributed to the different mode shape correction factors and modal combination methods used to account for the nonlinear mode shapes and lateral-torsional coupling, respectively.

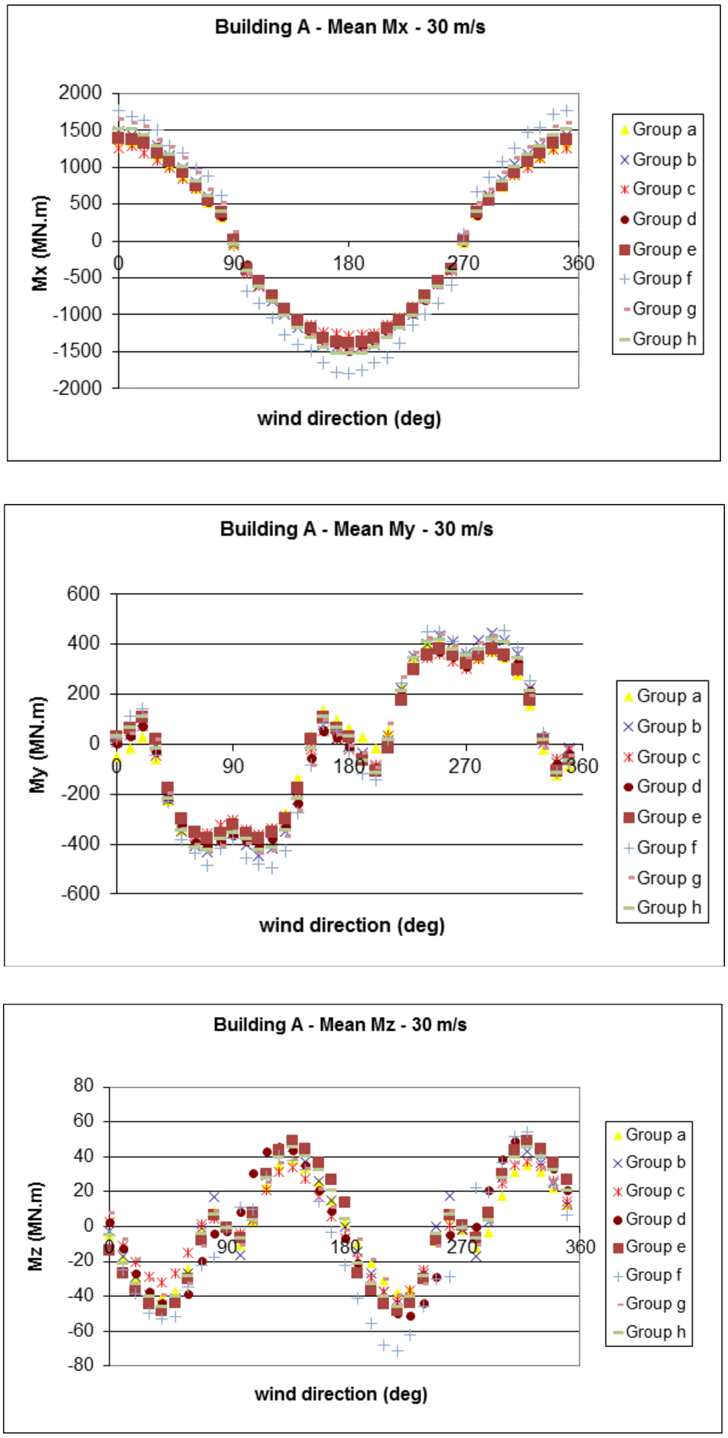

Fig. 12 Mean base moments ($\bar{U}_h$ = 30 m/s)

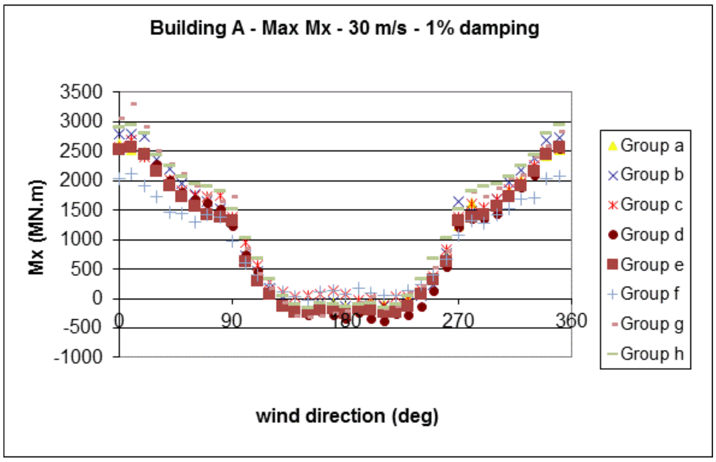

Fig. 13 Predicted maximum sway moments about the x-axis

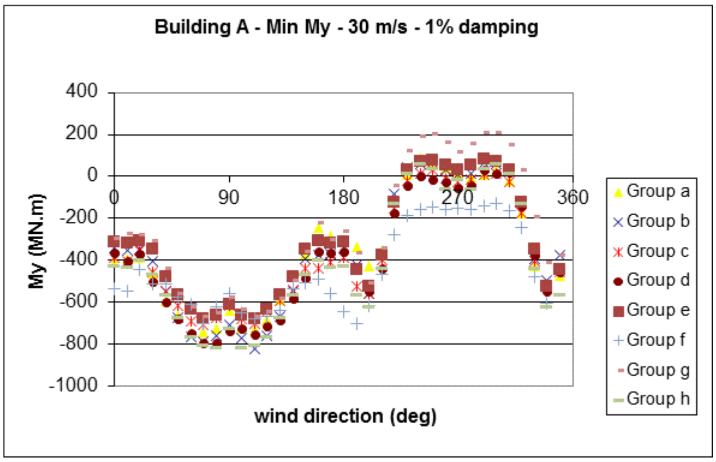

Fig. 14 Predicted minimum sway moments about the y-axis

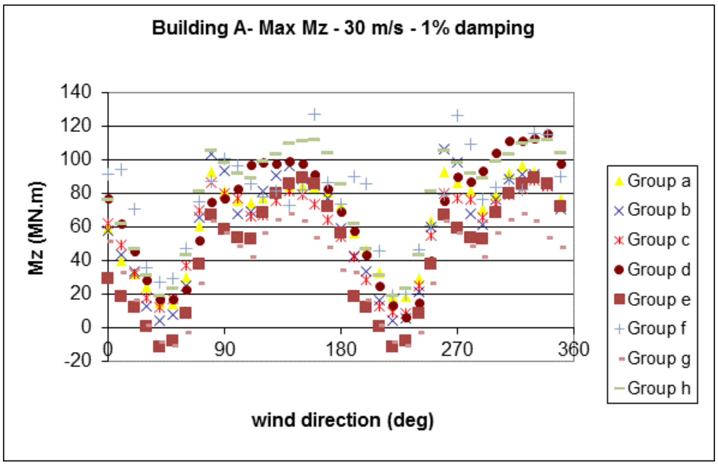

Fig. 15 Predicted maximum twist moments about the z-axis

3.2 Comparison of accelerations – Building A

Seven out of eight participating groups have provided resultant accelerations at the top of the building at the geometric centre, and at a corner of the roof. Figures 16 and 17 show predicted maximum accelerations at the centre and corner, respectively, for a mean wind speed at the top of the building of 30 m/s and critical damping ratio of 1%. Note that some acceleration results for Building A are still under review and are not included in the figures.

The predicted maximum accelerations are more scattered than for Building B, for both the centre, and corner, of the building; this can be attributed to the reasons stated in the preceding section, as well as the methods used for combining acceleration components. The predicted accelerations at the top of Building A are generally lower, in magnitude than those for Building B (compare Figure 17 to Figure 10). This is related to the higher frequencies of Building A, and the coupling of twist and sway in the modes.

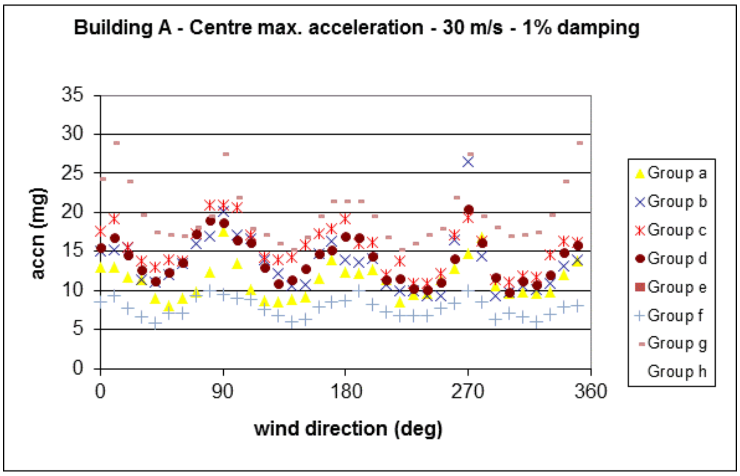

Fig. 16 Predicted centre accelerations for a mean wind speed of 30 m/s, 1% damping

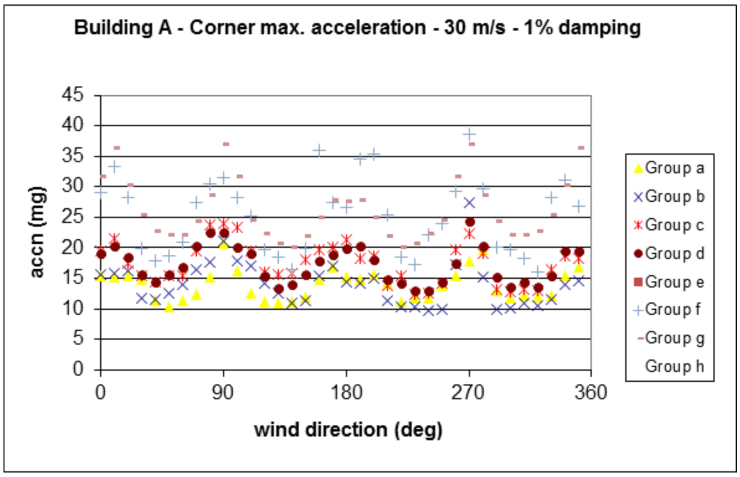

Fig. 17 Predicted corner accelerations for a mean wind speed of 30 m/s, 1% damping

4. CONCLUSIONS

Some comparisons from the International HFBB study have been presented in this paper. The results shown represent a small proportion of the total amount of material presented, but illustrate the main trends in the comparisons.

The mean and peak base moments from the participating groups show quite good agreement with coefficients of variation about the averages of 8-20% for Building B and 20-30% for Building A. The peak corner accelerations from five groups show greater variation, especially for the lowest wind speed, which is representative of a 1-year return period wind for many locations, and the results would therefore be available for comparison with occupant comfort criteria. The greater scatter in the acceleration predictions can be attributed to the dominance of the resonant components which rely on accurate measurement of spectra of generalized forces at relatively high reduced frequencies, a known difficulty with the HFBB technique. For buildings with nonlinear 3-dimensional mode shapes (i.e. lateral-torsional coupling), such as Building A in this study, the greater scatter in the predictions can be attributed to the usage of different mode shape correction factors, modal combination methods, and methods for combining acceleration components among the groups.

ACKNOWLEDGEMENTS

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APPENDIX A

The boundary-layer flow properties specified were similar for both buildings. They are summarized below:

Design mean wind speed at top of building (both cases): 20, 30 and 40 m/s (assumed uniform with wind direction)

Urban terrain. Mean velocity profile. Power law exponent: 0.25
(approximate roughness length z_0 : 0.2 mm)

Longitudinal turbulence intensity at rooftop of building B: 0.143
Longitudinal turbulence intensity at rooftop of building A: 0.131

Lateral and vertical intensities: not specified.

Integral turbulence length scale at rooftop height of building B: 175 m
Integral turbulence length scale at rooftop height of building A: 190 m