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Wind-induced response of structurally coupled twin tall buildings

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Abstract. The paper describes a study of the effects of structural coupling on the wind-induced response of twin tall buildings connected by a skybridge. Development of a dual high-frequency force balance used in wind tunnel investigation and background information on the methodology employed in analysis are presented. Comparisons of the wind-induced building response (rooftop acceleration) of structurally coupled and uncoupled twin buildings are provided and the influence of structural coupling is assessed. It is found that the adverse aerodynamic interference effects caused by close proximity of the buildings can be significantly reduced by the coupling. Neglecting of such interactions may lead to excessively conservative estimates of the wind-induced response of the buildings. The presented findings suggest that structural coupling should be included in wind-resistant design of twin tall buildings.

Keywords: wind-induced response; twin tall buildings; structural coupling; skybridge; dual high-frequency force balance; wind tunnel testing.

1. Introduction

The advent of innovative materials and construction technologies has led to design and construction of slender modern tall buildings, of reduced mass and damping. This trend has resulted in an increased susceptibility of such buildings to wind-induced excitation. To date, the aerodynamic performance of tall buildings has not been fully understood and no comprehensive analytical nor codified models have been developed to adequately address this topic. Accordingly, wind tunnel testing remains the only reliable tool used in fundamental and applied studies of wind effects on tall buildings.

Most of basic wind tunnel investigations of tall buildings have been focused on isolated buildings of generic shapes. However, many contemporary architectural designs involve two or more tall buildings of complex geometries, located in close proximity. In many cases, individual buildings are connected by one or more skybridges or skygardens located at various elevations and/or by ground-level podiums. In addition to the aerodynamic interference effects, caused primarily by close proximity of buildings, the aerodynamic performance of the buildings may be significantly affected by structural coupling due to inter-building connections. These effects increase complexity of wind-resistant design of such buildings. This in turn leads to challenges imposed on wind tunnel techniques needed to

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adequately model wind loading and, at times, to simulate the wind-induced building response.

A number of conventional wind tunnel techniques, e.g., the high-frequency force balance (HFFB) approach employing a single high-frequency force balance, have been used in wind engineering studies of tall buildings. However, typical applications of such tools do not allow for direct inclusion of the aerodynamic and structural coupling effects in predictions of the wind-induced response of connected buildings. As a result, the impact of structural linkages (coupling) on the aerodynamic performance of twin and multiple interconnected buildings has been frequently neglected or only approximately (indirectly) accounted for.

Advances in instrumentation render synchronous acquisition of data from multiple devices feasible. This feature can be utilized in nearly simultaneous acquisition of time series of wind loading from two or more high-frequency force balances. The obtained data can be used in investigations of complex dynamic responses of structurally connected twin buildings. A limited number of such studies have been reported in open literature, e.g., Xie and Irwin (1998, 2001) and Boggs and Hosoya (2001). In these efforts, the base aerodynamic loading was measured simultaneously for all major buildings of a multi-building complex involving structural coupling. Xie and Irwin (1998, 2001) employed such approach in a study of structurally connected twin buildings and have shown that the structural coupling effects led to equalized dynamic responses of the buildings. Boggs and Hosoya (2001) reported a study of a two-tower structure with a common podium, susceptible to coupled wind-induced motions. They measured the aerodynamic forces using two force balances mounted inside two isolated models of tall buildings and acquired a simultaneously sampled wind pressures acquired at a large number of pressure taps distributed on the located on the building surfaces. The obtained synchronized data were subsequently employed to calculate the wind-induced building response incorporating structural coupling. These techniques can be extended to building configurations comprising of a larger number of interconnected tall buildings. Overall, they represent a significant improvement in treatment of aerodynamic loading and structural coupling effects, for twin and multiple buildings.

Based on the experiences reported above and in other references, a dual-force-balance technique has been recently developed and applied in tall building investigations carried out at the Wind Engineering and Fluids Laboratory (WEFL), Colorado State University. This paper describes one of the studies - an investigation of the aerodynamic interference and structural coupling effects on generic tall buildings located in close proximity and connected by a skybridge. The paper is organized as follows. First, background information is presented. Next, the experimental configuration and instrumentation are described. The details of data acquisition and processing are provided. Finally, representative results illustrating the aerodynamic interference and structural coupling effects are presented. The findings of this study are summarized in a concluding section of the paper.

2. Background

The essence of the HFFB technique applied for linear structural systems is an experimental determination of base wind loading and calculation of the wind-induced structural responses. The analytical process involves the use of the modal superposition technique, which for an isolated tall building can be expressed as follows

$$u_s(z, t) = \sum_j q_j(t)\Phi_{js}(z) \tag{1}$$

where $u_s(z, t)$ is the s-component of the linear ($s = x$, y) or rotational about z-axis ($s = z$)

displacement of a building at the elevation z and time t, and $\Phi_{js}(z)$ is the s-component of the j-th mode shape. The principal coordinate $q_j(t)$ is determined from

$$m_j^* \ddot{q}_j(t) + c_j^* \dot{q}_j(t) + k_j^* q_j(t) = P_j^*(t) \tag{2}$$

where m_j^*, c_j^*, k_j^*, and $P_j^*(t)$ are, respectively, the generalized mass, damping, stiffness, and loading in the j-th mode. The generalized mass is $m_j^* = \sum_{s=x,y,z} \int_0^H \mu_s(z) \Phi_{js}^2(z)\, dz$, where $\mu_s(z)$ is the mass or mass moment of inertia per unit height, for respectively, the sway ($s = x$, y) or torsional ($s = z$) modal components, while z is the vertical ordinate and H is the building height. When the modes are assumed linear and uncoupled, the generalized loading $P_j^*(t)$ can be expressed in terms of the base aerodynamic overturning (sway) moment, Eq. (3a), or torque, Eq. (3b), which can be experimentally determined from the HFFB measurements.

$$P_j^*(t) = \int_0^H \Phi_j(z) p_j(z,t) dz = \int_0^H \left(\frac{z}{H}\right) p_j(z,t) dz = \frac{M_j(t)}{H} \tag{3a}$$

$$P_j^*(t) = \lambda M_j(t) \tag{3b}$$

where $p_j(z, t)$ is the externally applied loading (per unit height of a building); $M_j(t)$ is the external base aerodynamic moment or torque; λ is the empirical mode correction factor for generalized torsional loading, typically in the range of 0.5 to 0.7. In this paper, a representative value of $\lambda = 0.6$ was assumed.

The modal superposition technique, applied in analysis of wind-induced response of structurally coupled twin tall buildings, requires more than three modes of vibration. Investigations limited to only three modes are incapable of capturing the modal coupling effects, Xie and Irwin (2001), and Boggs and Hosoya (2001), and they may lead to an overestimation or underestimation of the wind-induced building responses. For a case of structurally connected two identical tall buildings of square plan – a twin building configuration – a set of six generic modes capable of capturing the coupling effects is depicted in Fig. 1.

The building motion, for such configuration, can be described by expanding Eq. (2), as follows

$$m_{cj}^* \ddot{q}_{cj}(t) + c_{cj}^* \dot{q}_{cj}(t) + k_{cj}^* q_{cj}(t) = P_{cj}^*(t) \tag{4}$$

where m_{cj}^*, c_{cj}^*, k_{cj}^*, $q_{cj}(t)$ and $P_{cj}^*(t)$ are, respectively, the coupled generalized mass, damping,

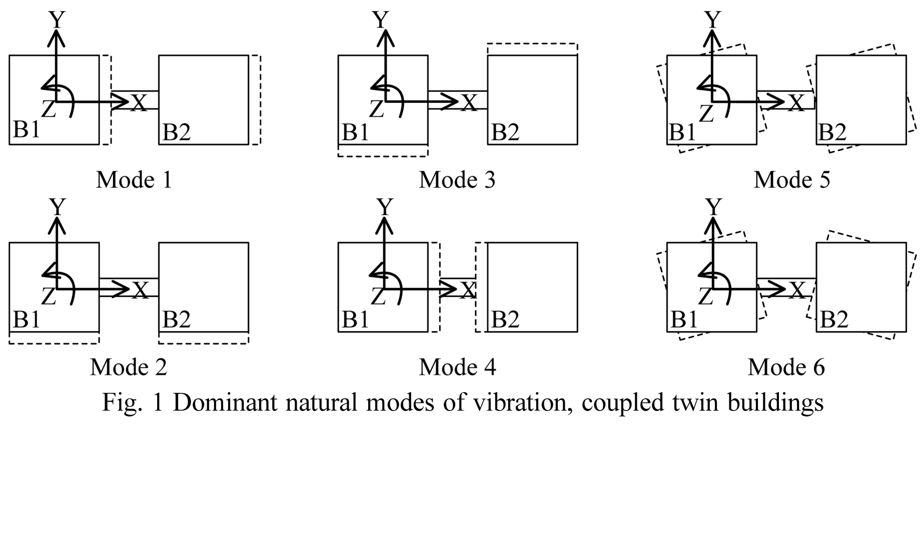


Fig. 1 Dominant natural modes of vibration, coupled twin buildings

stiffness, principal coordinate, and loading in the j-th mode; subscript c denotes a structurally coupled case. The coupled properties can be obtained via a superposition accounting for contributions from the two buildings. For example, the coupled generalized mass can be written as follows

$$m^*_{cj} = \sum_{n=1}^{2} \sum_{s=x,y,z} m^*_{jsn} \tag{5}$$

where m^*_{jsn} is the generalized mass in the j-th mode and s-direction ($s = x, y, z$), for building n ($n = 1, 2$).

The coupled generalized (modal) loading $P^*_{cj}(t)$, in Eq. (4), can be expressed in terms of the modal loading $P^*_{sn}(t)$ along axis s on building n, simultaneously acquired using the dual high-frequency force balance system, and the directional mode correction coefficients η_{jsn}.

$$P^*_{cj}(t) = \sum_{n=1}^{2} \sum_{s=x,y,z} \eta_{jsn} P^*_{sn}(t) \tag{6}$$

where

$$P^*_{sn}(t) = \sum_{i=1}^{N} p_{isn}(t) \Phi_{ijsn} \tag{7}$$

where $p_{isn}(t)$ is the externally applied loading at the i-th floor, along s-axis, for building n; Φ_{ijsn} is the modal shape at the i-th floor, in the j-th mode and s-direction, for building n; and N is the number of floors, assumed to be the same for the two buildings.

Eq. (6) can be expanded as follows

$$\begin{Bmatrix} P^*_{c1}(t) \\ P^*_{c2}(t) \\ P^*_{c3}(t) \\ P^*_{c4}(t) \\ P^*_{c5}(t) \\ P^*_{c6}(t) \end{Bmatrix} = \begin{bmatrix} \eta_{1x1} & \eta_{1y1} & \eta_{1z1} & \eta_{1x2} & \eta_{1y2} & \eta_{1z2} \\ \eta_{2x1} & \eta_{2y1} & \eta_{2z1} & \eta_{2x2} & \eta_{2y2} & \eta_{2z2} \\ \eta_{3x1} & \eta_{3y1} & \eta_{3z1} & \eta_{3x2} & \eta_{3y2} & \eta_{3z2} \\ \eta_{4x1} & \eta_{4y1} & \eta_{4z1} & \eta_{4x2} & \eta_{4y2} & \eta_{4z2} \\ \eta_{5x1} & \eta_{5y1} & \eta_{5z1} & \eta_{5x2} & \eta_{5y2} & \eta_{5z2} \\ \eta_{6x1} & \eta_{6y1} & \eta_{6z1} & \eta_{6x2} & \eta_{6y2} & \eta_{6z2} \end{bmatrix} \begin{Bmatrix} P^*_{x1}(t) \\ P^*_{y1}(t) \\ P^*_{z1}(t) \\ P^*_{x2}(t) \\ P^*_{y2}(t) \\ P^*_{z2}(t) \end{Bmatrix} \tag{8}$$

For an idealized case of twin square buildings and six modal shapes shown in Fig. 1, Eq. (8) can be significantly simplified if the modes are assumed to be linearly dependent on the vertical ordinate z. After incorporating Eq. (3) in Eq. (8) the time series of the coupled modal loading $P^*_{cj}(t)$ can be expressed in terms of the time series of the measured sway and torsional base moments M_{sn}.

$$\begin{Bmatrix} P^*_{c1}(t) \\ P^*_{c2}(t) \\ P^*_{c3}(t) \\ P^*_{c4}(t) \\ P^*_{c5}(t) \\ P^*_{c6}(t) \end{Bmatrix} = \begin{bmatrix} 1 & 0 & 0 & 1 & 0 & 0 \\ 0 & 1 & 0 & 0 & 1 & 0 \\ 0 & 1 & 0 & 0 & -1 & 0 \\ 1 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & \lambda & 0 & 0 & \lambda \\ 0 & 0 & \lambda & 0 & 0 & -\lambda \end{bmatrix} \begin{Bmatrix} M_{y1}(t)/H \\ M_{x1}(t)/H \\ M_{z1}(t) \\ M_{y2}(t)/H \\ M_{x2}(t)/H \\ M_{z2}(t) \end{Bmatrix} \tag{9}$$

Subsequently, the wind-induced modal accelerations of structurally coupled twin buildings can be computed, in a manner similar to that for an isolated building. Using white noise approximation, the standard deviation of the resonant modal acceleration is given by

$$\sigma_{\ddot{q}_{cjr}} = \frac{1}{m_{cj}^{*}} \sqrt{\frac{\pi f_{cj} S_{P_{cj}^{*}}(f_{cj})}{4\zeta_j}}. \tag{10}$$

where f_{cj} is the coupled modal frequency in the j-th mode; $S_{P_{cj}^{*}}(f_{cj})$ is the power spectral density of the coupled modal loading; and ζ_j is the structural damping ratio. The modal contribution, in the j-th mode and s-direction, to the peak rooftop acceleration of the n-th building is given by

$$\hat{a}_{jsn} = g_j \sigma_{\ddot{q}_{cjr}} \Phi_{jsn} \tag{11}$$

where g_j is the peak factor in the j-th mode.

To determine the coupled wind-induced acceleration for primary structural directions, $\hat{a}_{cs}$, from the calculated modal peak wind responses, the modal combination including cross-modal coupling is employed. In the present study, the CQC (complete quadratic combination), Wilson *et al.* (1981), is applied and the directional peak accelerations are computed as follows

$$\hat{a}_{cs} = \sqrt{\sum_{j=1}^{6} \sum_{m=1}^{6} \hat{a}_{jsn} \hat{a}_{msn} \rho_{jm}} \tag{12}$$

where a_{jsn} and a_{msn} are, respectively, the modal peak accelerations of the n-th building, associated with the s-direction of the j-th and m-th modes. The cross-modal coefficient ρ_{jm} between the j-th and m-th modes is expressed as

$$\rho_{jm} = \frac{8\zeta^2(1 + r_{jm})r_{jm}^{1.5}}{(1 - r_{jm}^2)^2 + 4\zeta^2 r_{jm}(1 + r_{jm})^2} \tag{13}$$

where ζ is the structural damping ratio and r_{jm} denotes the frequency ratio, f_{cj}/f_{cm}.

Finally, total peak acceleration at roof-top corner of the building is determined

$$\hat{a}_{total} = \sqrt{\hat{a}_{cx}^2 + \hat{a}_{cy}^2 + \hat{a}_{cz}^2 - \sqrt{2}\hat{a}_{cx}\hat{a}_{cz} + \sqrt{2}\hat{a}_{cy}\hat{a}_{cz}} \tag{14}$$

3. Experimental details and data processing

3.1. Dual force balance system and models

The dual force balance system employed in the present study consisted of two high-frequency six-component force balances, and two data acquisition boards coupled by a synchronizing cable installed in a personal computer. This arrangement allowed for synchronized acquisition of the data from the two balances, needed for evaluation of the structural coupling effects on the wind induced response of tall twin buildings. Light-weight models of tall buildings arranged in various twin-building configurations were attached to the balances. A modular support of the models and the force balances was developed to sustain high resonant frequency of the system and to facilitate versatile adjustment in the relative position of and spacing between the models. A close-up view of

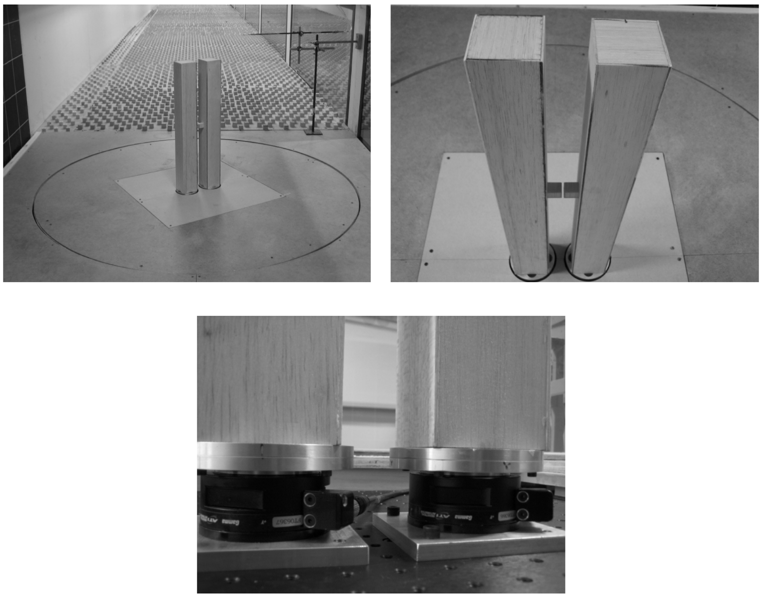


Fig. 2 Representative twin-building model and force balance system

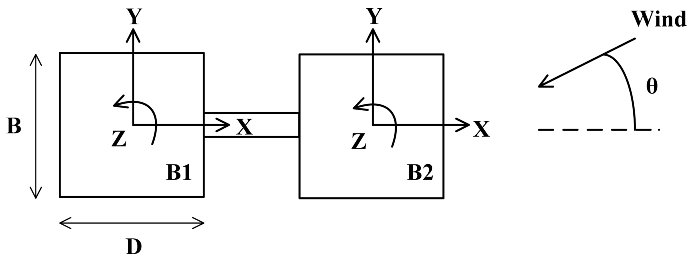


Fig. 3 Coordinate systems and wind direction

the balances and the support system, as well as overall views of the model of the tested twin-building configuration with a skybridge are shown in Fig. 2.

Twin buildings of fixed square plan and five aspect ratios (height-to-planar dimension), ranging from 4 through 8, were considered. The geometrical scale was 1 : 500. The models were 7.6 cm x 7.6 cm in cross section and they were made of thin sheets of balsa wood. The spacing between the adjacent (parallel) facades of the models was kept constant, 5 cm. A skybridge located at the building mid-height was modeled using two pieces of solid balsa wood. Each piece was attached to

Table 1 Geometrical properties of prototype buildings

Building model	Planar dimension (m)	Height (m)	Side ratio	Aspect ratio
TBS4	38	152	1	4
TBS5	38	191	1	5
TBS6	38	229	1	6
TBS7	38	267	1	7
TBS8	38	305	1	8

one of the (two) model buildings and a small gap between these pieces was maintained to ensure acquisition of unbiased wind loading exerted on each model. An overall view of a representative twin-building model, mounted on the dual-force balance system and installed in the Meteorological Wind Tunnel (MWT) at WEFL, Colorado State University, is included in Fig. 2.

For reference (in discussion of the results), one of the buildings was denoted the primary building and was labeled as B1, while the remaining building was labeled B2. This notation and the definitions of the coordinate systems and the wind direction are displayed in Fig. 3. The geometrical parameters of the considered prototype tall buildings are listed in Table 1.

3.2. Approach flow

Wind tunnel testing was carried out in the MWT, in the ABL (atmospheric boundary layer) flow representative for suburban wind exposure (with power exponent $\alpha = 0.21$), modeled at a 1:500

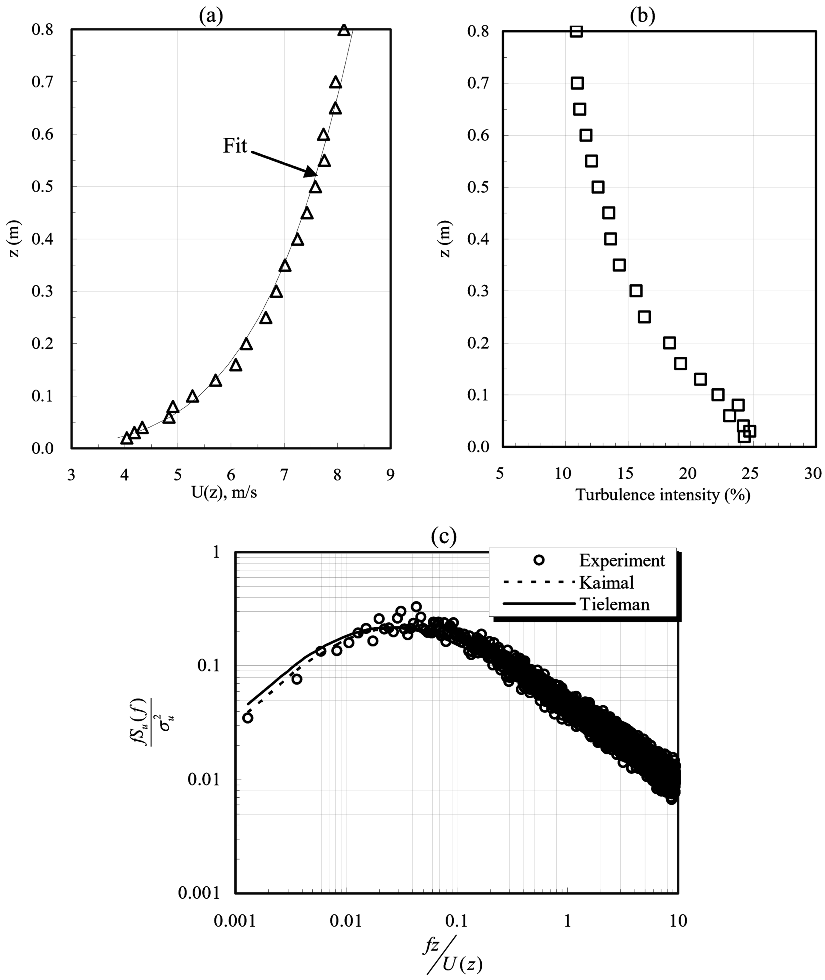


Fig. 4 Simulated approach flow: (a) mean wind velocity, (b) along-wind turbulence intensity and (c) along-wind velocity spectrum at prototype elevation z = 50 m

geometrical scale. Passive devices (spires and barriers) similar to those used in past studies of tall buildings, carried out at WEFL, were used in combination with a long upstream fetch of floor roughness. Fig. 4 shows the mean velocity and the along-wind turbulence intensity profiles, and the along-wind velocity spectrum at the prototype height of 50 m, acquired immediately upstream of the model, with the model removed from the turn-table. The measured spectrum is compared with the empirical velocity spectra proposed by Kaimal and Tieleman.

3.3. Data acquisition and analysis

The wind-induced base loadings – the overturning (sway) and torsional moments and shear forces – were acquired for the two buildings at 19 wind directions, with a 10^o- interval in the wind azimuth. The resonant frequency of the force balance with an attached building model varied, depending on the model height. Its lowest value (corresponding to the tallest model) was 86 Hz. A typical record of the acquired data comprised of 16384 data points per segment, sampled at 2000 samples per second. The corresponding full-scale record length of the data was 10 minutes. Thirty six segments of the data were acquired for each wind direction. Segment and two-point frequency averaging was carried out during calculation of the wind loading power spectra. As a result, the normalized spectral error, Bendat and Piersol (2000), was approximately equal to 12%.

The data analysis discussed in Sec. 2 was employed to process the measured aerodynamic loading and to determine the coupled dynamic responses of the considered twin buildings. Wind-induced rooftop corner accelerations associated with the assumed building damping ratio of 1.5%, gross mass density of 200 kg/m^3, and 1-year return period were determined.

For comparison, an uncoupled case was also considered. For this case, three uncoupled modes, linearly dependent on the vertical ordinate z were assumed. Two modes were translational (in x and y directions) and the third mode was torsional (about z-axis). It should be noted that a set of six idealized (linear) modes of vibration, shown in Fig. 1, was employed in the coupled case.

For the uncoupled cases, the fundamental natural frequencies listed in Table 2 were determined using

Table 2 Natural frequencies of uncoupled vibration of prototype buildings (Hz)

Building model	f_x	f_y	f_z
TBS4	0.3	0.3	0.45
TBS5	0.25	0.25	0.37
TBS6	0.21	0.21	0.31
TBS7	0.18	0.18	0.27
TBS8	0.16	0.16	0.24

Table 3 Natural frequencies of the lowest six modes of coupled prototype tall buildings (Hz)

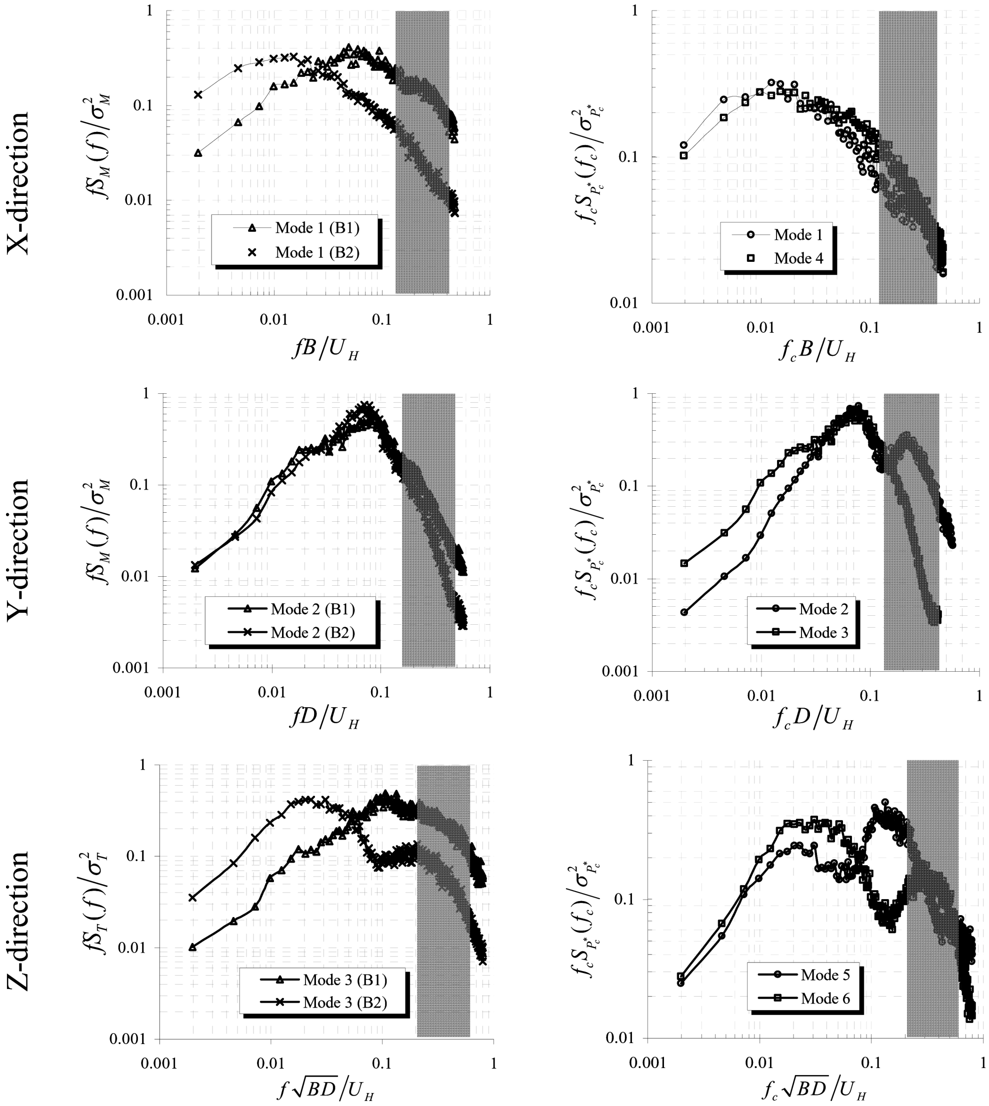
Model	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6
TBS4	0.3	0.31	0.33	0.38	0.45	0.5
TBS5	0.25	0.26	0.28	0.33	0.37	0.4
TBS6	0.21	0.22	0.24	0.27	0.31	0.34
TBS7	0.18	0.19	0.21	0.24	0.27	0.29
TBS8	0.16	0.17	0.19	0.22	0.24	0.26

empirical formulae proposed by Lagomarsino (1993). Based on feedback received from structural engineering consultants, these frequencies were adjusted to account for structural coupling, see Table 3, and the adjusted values were used in the response calculations carried out for the coupled case.

4. Results and discussion

4.1. Comparison of modal wind loading spectra

Representative normalized power spectra of the modal aerodynamic loading, for a twin-building



(a) (b)

Fig. 5 Modal normalized power spectra of wind loading for TBS8, wind direction 0°: (a) uncoupled and (b) coupled cases configuration comprising of buildings of the 8:1 aspect ratio (denoted TBS8 in Table 1), are shown in Fig. 5, wind direction of 0° (see Fig. 3). The results for the uncoupled, Fig. 5(a), and coupled, Fig. 5(b), cases are presented. Of principal interest in this comparison were the loading spectra in the frequency range of importance in serviceability analyses, shown shaded in Fig. 5.

In this range, the uncoupled spectra for the along-wind (x) and torsional (z) loading components of the downwind building B1 are higher than those for the upwind building B2. This difference is attributed to unsteadiness in the wake generated by the upstream building B2. In the cross-wind (y) direction, the spectra on the two buildings are approximately the same.

A similar comparison carried out for the coupled case reveals that the coupled spectra in the along-wind (*x*) and torsional (*z*) directions are approximately the same spectral levels of the two contributing modes – 1 and 4 for x-direction, and 5 and 6 for z-direction. In the cross-wind (y) direction, the spectrum is higher for mode 2, associated with the in-phase motion, than that for the “out-of-phase” mode 3.

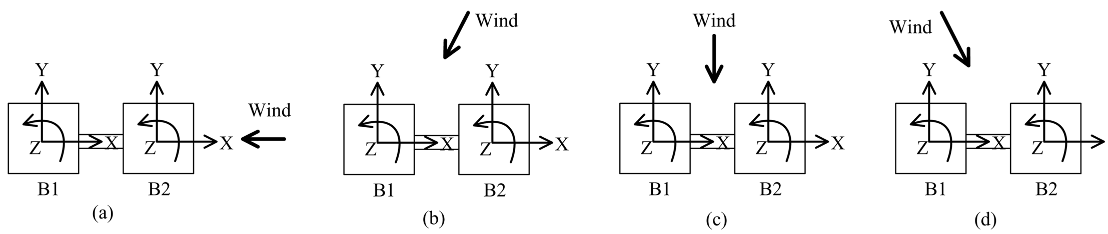


Fig. 6 Critical wind directions: (a) 0°, (b) 60° ~ 80°, (c) 90° and (d) 100° ~ 120°

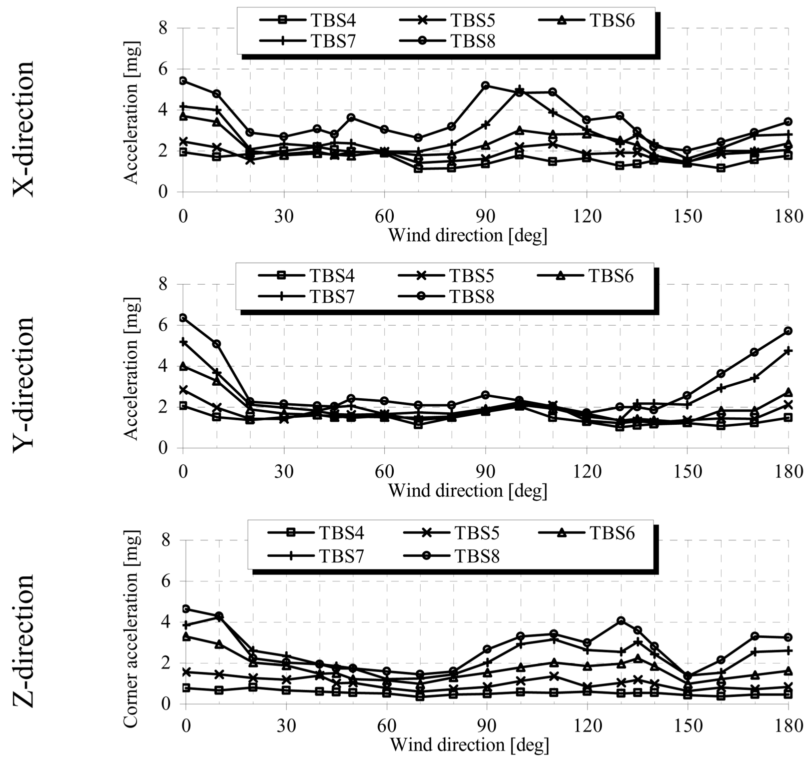


Fig. 7 Peak accelerations of B1, uncoupled case

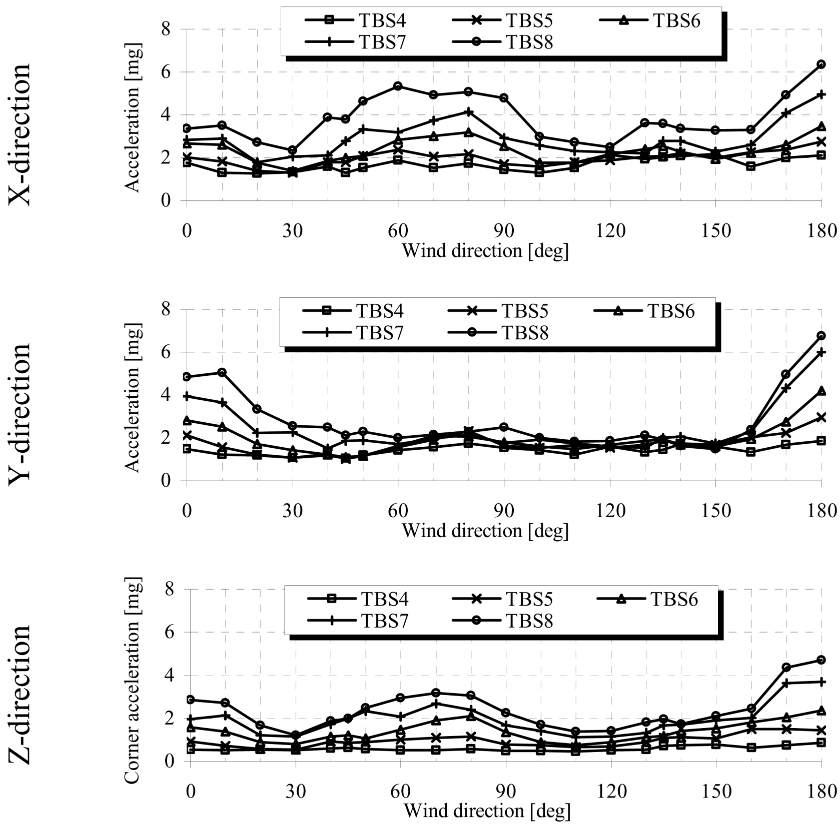


Fig. 8 Peak accelerations of B2, uncoupled case

4.2. Uncoupled response

Four critical wind directions identified in analysis of the wind-induced top-floor accelerations of the considered twin-building configurations are indicated in Fig. 6, cases (a) through (d). The peak accelerations of uncoupled buildings B1 and B2 are presented in Figs. 7 and 8, respectively. In case (a) – wind aligned with the twin buildings – no shielding effects are exhibited and the accelerations of the downwind building B1 are higher than those of the upwind building B2. For cornering winds – cases (b) and (d) – the accelerations of the upwind building are higher than those of the downwind building – by up to 50% in the x- and z-directions. For the normal wind direction – case (c) – the peak accelerations of the two buildings are very similar. These results are in agreement with the aerodynamic interference effects reported by Bailey and Kwok (1985), Huang and Gu (2005), and others.

The above findings indicate that in absence of structural coupling the wind-induced responses of the two buildings are different and this disparity depends on the wind direction and on the aspect ratio.

4.3. Coupled response

Fig. 9 shows modal contributions to the peak acceleration of the building roof corner. It can be seen that, depending on the wind direction, either the modal peak responses generated by in-phase building motions (the first, second, and fifth modes) or those associated with out-of-phase motions (the third, fourth, and sixth modes) are predominant. For wind aligned with the twin buildings (wind direction of 0 and 180 degrees), the dominant contribution of the second mode (in cross-wind direction) is clearly displayed in Fig. 9. It can be seen that this modal participation significantly increases with a rise in the building aspect ratio.

The modal peak accelerations in Fig. 9 were subsequently used to calculate the directional top-

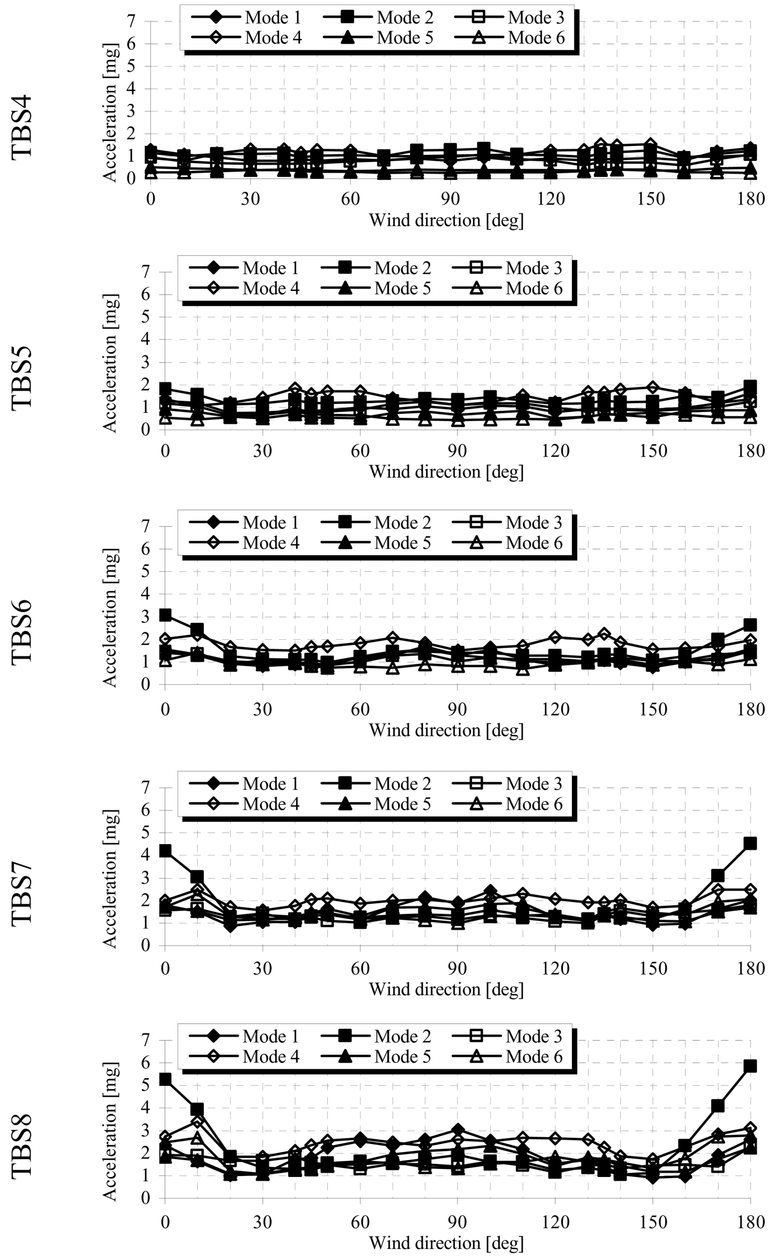


Fig. 9 Modal contributions to peak acceleration, coupled case

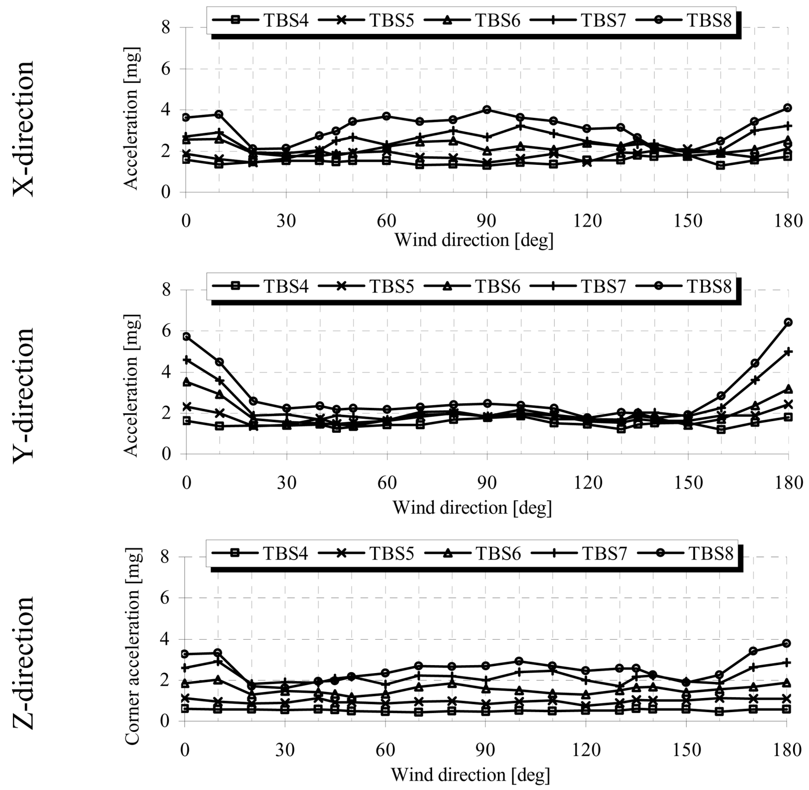


Fig. 10 Peak accelerations, coupled case

floor peak accelerations (as discussed in Sec. 2), see Fig. 10. As expected, the accelerations increase with the building aspect ratio and they are dependent on the wind direction. All the acceleration components exhibit large values in vicinity of wind directions of 0° and 180°. This is in contrast with small magnitudes occurring in vicinity of wind directions of 30° and 150°. For the remaining wind directions, the acceleration magnitudes of *x*- and *z*-components are moderate large and they exhibit strong dependence on the aspect ratio. In the *y*-direction, they are smaller and less dependent on the aspect ratio.

4.4. Comparison of coupled and uncoupled responses

A comparison of the results obtained for the coupled and uncoupled cases is presented in Figs. 11 through 13. The coupled-to-uncoupled response ratios are shown in Figs. 11 and 12, respectively for buildings B1 and B2. It can be seen that for wind directions associated with significant aerodynamic interference effects – cases (a), (b) and (d) in Fig. 6 – the peak accelerations are reduced (in one or more components) by up to 30%, in presence of structural coupling. For the wind direction of 90°, case (c), the coupled and uncoupled peak accelerations are similar, in all the components.

The overall effects of structural coupling on the total peak accelerations of the top-floor roof corner can be inferred from Fig. 13. It is shown that the largest wind-induced response – building B1 at the wind direction of 0° and building B2 at 180° – can be reduced by up 30%. This reduction is accompanied by an increase in the response of the companion building. As a result, the response

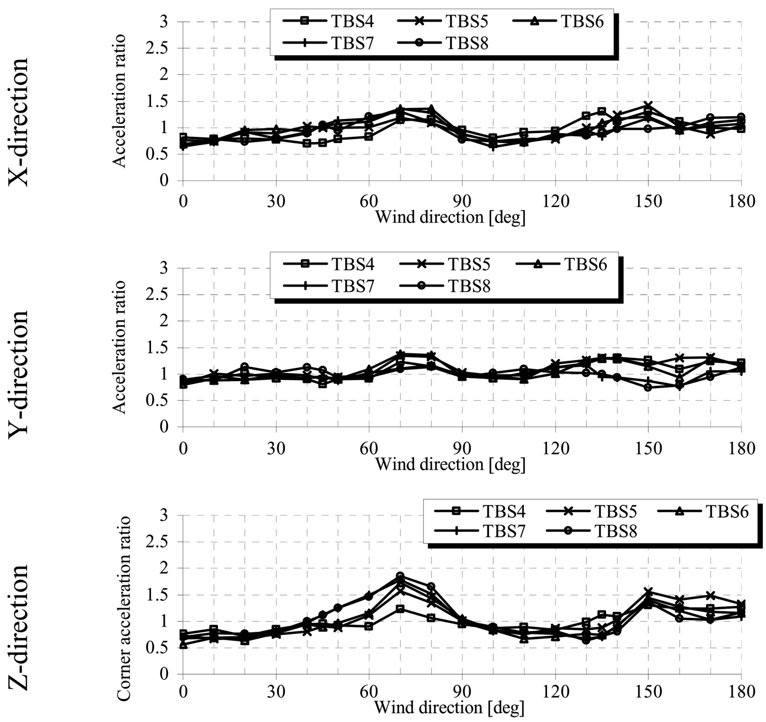


Fig. 11 Coupled-to-uncoupled acceleration ratios, building B1

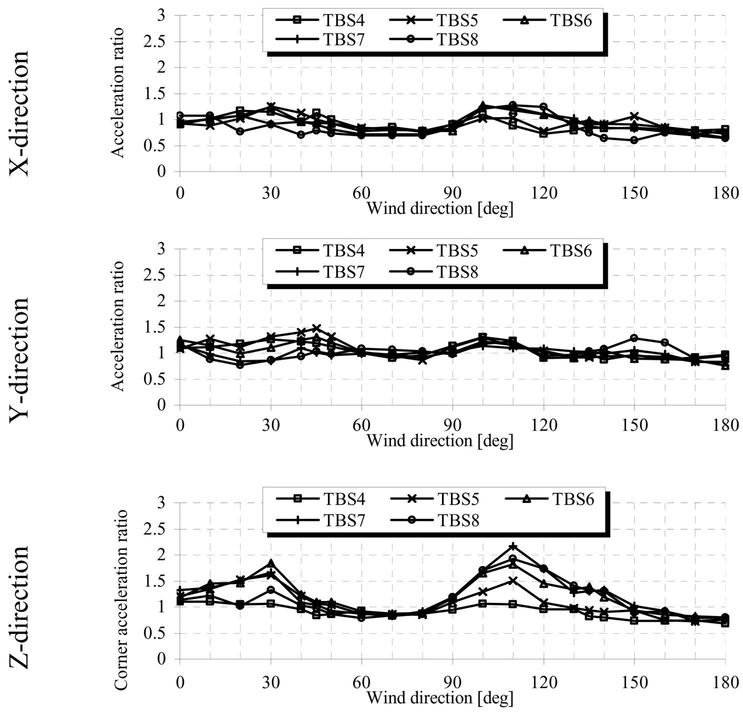


Fig. 12 Coupled-to-uncoupled acceleration ratios, building B2

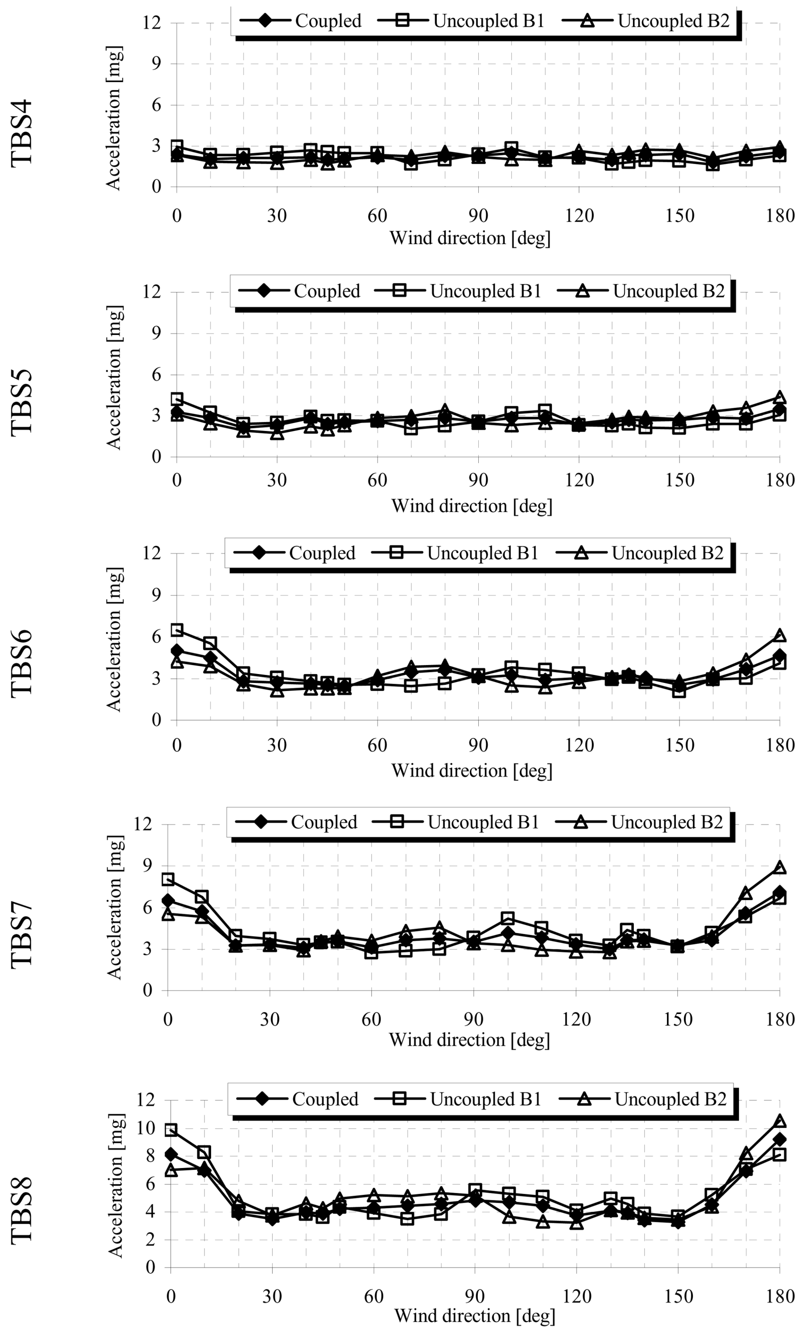


Fig. 13 Comparisons of total peak accelerations, top-floor corner

equalization is achieved and the largest response is significantly reduced when structural coupling is included. These findings are in agreement with the results for tall twin buildings with a skybridge, reported by Xie and Irwin (1998, 2001).

5. Concluding remarks

The findings of this investigation can be summarized as follows:

(1) The aerodynamic response – peak acceleration of top floor corner – of twin buildings was significantly affected by aerodynamic interference and structural coupling. These effects were dependent on the wind direction and building aspect ratio.

(2) In absence of structural coupling, the top floor accelerations were the largest for the downwind building and wind aligned with the twin buildings. For cornering winds, the upwind building exhibited a moderately bigger response than the downstream building. Small differences in the responses were observed for wind normal to the twin buildings.

(3) Structural coupling of buildings in twin arrangement (with skybridge) led to equalization of the response of the buildings. Effectively, the largest response of the buildings was reduced by approximately 30%. These observations are in agreement with findings reported by Xie and Irwin (1998, 2001).

(4) In view of significant potential benefits, structural coupling should be taken into account in wind-resistant design of twin tall buildings.

In practice, inclusion of structural coupling increases complexity of design. Systematic fundamental and applied studies are needed to improve the understanding of this subject and to aid development of optimized framework for wind-resistant design of twin tall buildings and other building complexes involving structural coupling.

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