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Wind-induced response of a twin-tower structure

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Abstract. With a newly developed multi-force-balance system(MFB), a twin-tower structure was studied for its wind-induced responses. The MFB system allowed the twin towers, which were linked structurally, to be treated as a single structural system with its corresponding modes of vibration involving coupled motions of the two towers. The towers were also studied using a more conventional force balance approach in which each tower was treated as an isolated structure, i.e., as though no structural link existed. Comparison of the results reveals how the wind loads between the towers are redistributed through the structural links and the modal couplings. The results suggest that although the structural links usually have beneficial impacts on wind-induced response, they may also play a negative role if the frequency ratios of pair modes are near 1.0.

Key words: wind-induced twin-tower response; multi-force-balance (MFB); high-frequency force-balance (HFFB); wind tunnel tests.

1. Introduction

When several buildings are located near by, they are sometimes designed to be structurally linked with each other. The structural links could be in various forms, such as skybridges or a common podium structure. Although the wind response is not the only consideration for deciding whether the nearby buildings should be structurally connected, the structural connections do have impacts on wind-induced response. In addition to the aerodynamic effects due to interactive wind flow around buildings, the structural links induce extra structural dynamic effects on each tower. Due to aerodynamic effects, the downwind tower may experience either shelter effects or wake buffeting effects from the upwind tower, and the upwind tower may be affected by increased or decreased wind loads due to interactive flow, if these two towers are close enough. The structural dynamic effects generally tend to equalize the wind-induced response among each individual tower by transferring the kinetic energy from a higher energy zone to a lower energy zone.

From the design point of view, structural engineers are concerned about (1) whether the structural links between towers will increase or decrease the wind-induced response (such as loads and accelerations) on each individual tower, and (2) when considering the overall structural response (such as overall overturning moments at base level), what correlation and phase should be considered for wind loading on each tower. The second issue is a particularly interesting in case when there is an expansion joint between the towers and the wind-induced relative deflections are of concern.

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A newly developed multi-force-balance system (MFB) in RWDI's boundary layer wind tunnel provides a practical solution to deal with wind-induced response on structurally connected multi-towers (Xie & Irwin 1998). The theoretical background of this system will be briefly reviewed in this paper.

To better understand the wind-induced response on structurally connected towers, a typical twin-tower structure was selected for investigation. With the MFB system, the twin towers were tested in RWDI's 1.9 m × 2.4 m boundary wind tunnel. The results for the structurally linked twin-towers were compared with a more conventional case where there were no structural links between the two towers. With the importance of structural links on the wind response being confirmed, it was found that the tower motions in opposing directions could be as significant as the motion in the same direction. The opposing motions could be further enhanced under certain surrounding conditions. It was also found that the frequency ratio of pair modes was an important parameter for wind-induced dynamic response. The definition of pair modes is given in Section 3.1. At a higher frequency ratio, the wind-induced dynamic response seems to be better equalized between the two towers. However, at a low frequency ratio, both towers may experience the worst case response, which is unfavourable from the point of view of wind-resistance design.

2. MFB System

With the MFB system, a tower complex is divided into several substructures. Each tower, treated as an individual substructure, is mounted on an individual high-frequency force-balance. The links between the substructures are disconnected during tests to ensure only the wind loads on each substructure are measured by the corresponding force balance, as shown in Fig. 1. Except that these towers have to be tested simultaneously, the MFB method is basically the same as the traditional force balance approach for wind tunnel testing. The main efforts in the MFB method are focussed on analysis using simultaneously measured data on each tower to account for their structurally linked nature.

Due to structural links, any mode of vibration will involve the motion of each tower. For example, the j-th mode shape will generally be

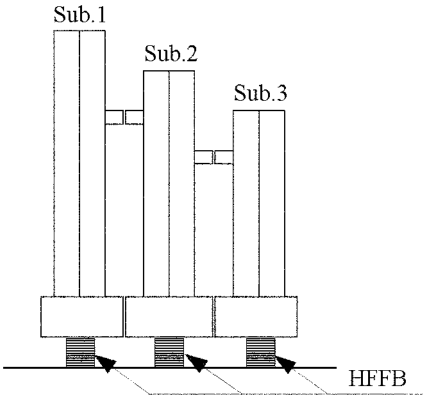

Fig. 1 Illustration of MFB system

$$\{\Phi_j(z)\} = \begin{Bmatrix} \Phi_{j1}(z) \text{ of Tower } 1 \\ \Phi_{j2}(z) \text{ of Tower } 2 \\ \vdots \\ \Phi_{jn}(z) \text{ of Tower } n \end{Bmatrix} \tag{1}$$

The generalized mass of the entire structure for the j-th mode can be obtained by the summation of the contributions from each tower, i.e.,

$$M_j = \sum_{k=1}^{n} M_{jk} \tag{2}$$

Similarly, the generalized force of the entire structure for the j-th mode can be obtained by the summation of the contribution from each tower, i.e.,

$$P_j(t) = \sum_{k=1}^{n} P_{jk}(t) \tag{3}$$

By assuming the effective fluctuating components of the approaching wind pressure to be linearly distributed over the height, the contribution of the generalized force from each tower can be obtained from the measured base overturning moments, base shears and base torsion in MFB testing. In general, the contribution from the k-th tower to the j-th mode is as follows :

$$P_{jk}(t) = (\Upsilon_{jFx} + \Lambda_{jFx})F_{kx}(t) + (\Upsilon_{jFy} + \Lambda_{jFy})F_{ky}(t) + (\Upsilon_{jMy} + \Lambda_{jMy})\frac{M_{ky}(t)}{H} + (\Upsilon_{jMx} + \Lambda_{jMx})\frac{M_{kx}(t)}{H} + \Upsilon_{jMz}\frac{M_{kz}(t)}{r} \tag{4}$$

where Υ_{iFx} = contribution factor of shear force F_{kx} due to sway motion; Υ_{jFy} = contribution factor of shear force F_{ky} due to sway motion; Υ_{jMy} = contribution factor of bending moment M_{ky} due to sway motion; Υ_{jMx} = contribution factor of bending moment M_{kx} due to sway motion; Υ_{jMz} = contribution factor of torsional moment M_{kz} due to torsional motion; Λ_{jFx}= contribution factor of shear force F_{kx} due to offset of torsional motion; Λ_{jFy} = contribution factor of shear force F_{ky} due to offset of torsional motion; Λ_{jMy} = contribution factor of bending moment M_{ky} due to offset of torsional motion; Λ_{jMx} = contribution factor of bending moment M_{kx} due to offset of torsional motion; H = building height; r = typical distance for normalizing torsional mode shapes. The formulae for these contribution factors were given by Xie and Irwin in 1998. These factors take into account nonlinear mode shapes and offsets between the centre of mass, centre of stiffness and the geometric centre.

With generalized mass and generalized force available, the wind-induced structural response of the entire structure can be calculated using buffeting theory, The dynamic responses on each tower are then obtained with proper modal combinations. In the present study, the complete quadratic combination (CQC) method was employed (Wilson 1981). With this method, the total response R is given by the following double summation over N modes :

$$R = \sqrt{\sum_{i=1}^{N}\sum_{j=1}^{N} R_i \rho_{ij} R_j} \tag{5}$$

where R_i and R_j are the responses due to the i-th mode and the j-th mode, respectively. ρ_{ij} is called the cross-modal coefficient between mode i and mode j and is given as follows :

$$\rho_{ij} = \frac{8\zeta^2(1+r_{ij})r_{ij}^{3/2}}{(1-r_{ij}^2)^2 + 4\zeta^2 r_{ij}(1+r_{ij})^2} \tag{6}$$

where r_{ij} = frequency ratio between mode i and mode j ; ζ = damping ratio.

3. Experiments on Twin Towers

The studied building complex consists of three towers, as shown in Fig. 2. The highest tower (on the left side of the photo), defined as Tower A, was a 43-story office tower with total height of 171 m. The other two identical towers were 46-story residential towers with total height of 156 m. The middle tower was defined as Tower B and the other Tower C. The Tower A was structurally isolated from Tower B and C, while Tower B and Tower C were structurally linked below podium level. The study was focussed on the wind-induced response of the structurally linked twin towers, Tower B and Tower C.

The 1:300 scale models of Tower B and Tower C were mounted on two force balances and tested simultaneously in RWDI's 1.9 m × 2.4 m boundary wind tunnel. The far-field approaching wind profile was similar to suburban for all directions. The wind direction was defined in degrees measured clockwise from north. The coordinates assigned to the twin towers for analysis are shown in Fig. 3, where the y axis is offset from the north by 10°. In Fig. 3, the left side tower represents Tower B and the right side tower represents Tower C.

The full scale mean and fluctuating loads of each tower, including overturning moments, shears and torsion, were determined by applying scaling factors to the model Loads measured by 5-

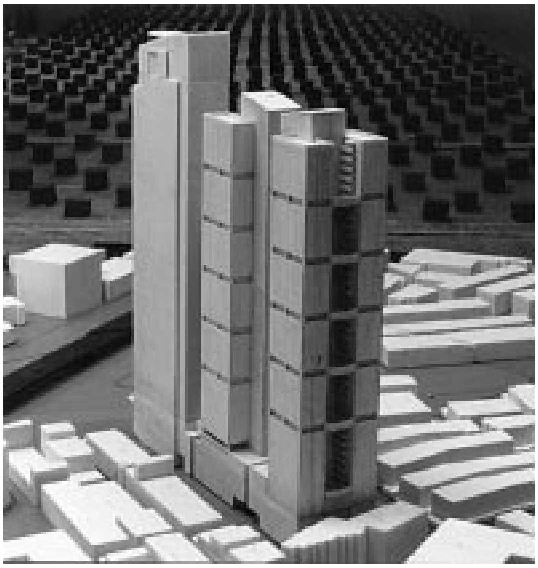

Fig. 2 1:300 scale wind tunnel models

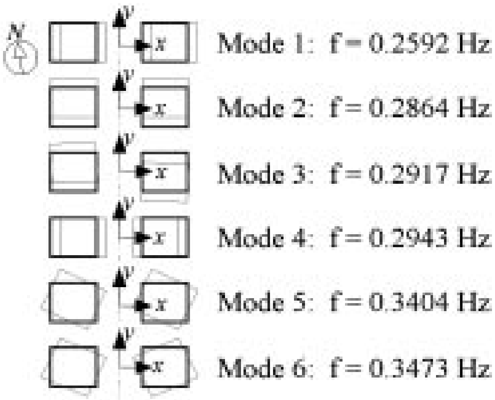

Fig. 3 The lowest six modes of the twin towers

component force balances. The generalized forces were calculated using Eq. (4). By solving the equation of motion based on the determined generalized forces and generalized masses, the accelerations and inertial loads were calculated as a function of wind speed and wind direction.

4. Results

4.1. Modes of vibrations of twin towers

Fig. 3 illustrates the first 6 modes of vibration for the twin towers. As shown in this figure, the modes for a twin tower structure are typically presented in pairs. For example, Mode 1 and Mode 4 are both *x*-direction sway modes, but Mode 1 is in the same direction for the two towers and Mode 4 is in the opposing direction for the two towers. So Mode 1 and Mode 4 are considered as a pair of *x*-direction modes. Similarly, Mode 2 and Mode 3 are a pair of *y*-direction modes, and Mode 5 and Mode 6 are a pair of torsional modes. It will be shown later that the frequency ratio of the pair modes tends to be an important parameter for dynamic response due to cross-modal correlations. This frequency ratio is generally determined by the configuration and stiffness of the structural links.

To investigate the impacts of structural links on the wind-induced responses, a hypothetical case was considered where the Tower B and Tower C were structurally isolated. Due to their being structurally identical, Tower B and Tower C had the same set of modes. The lowest frequency for *x*-direction motion in this case was close to the frequency of Mode 1 of the corresponding structurally linked structure (i.e., 0.2592 Hz). To simplify the comparison, the frequencies of the interesting modes of vibration for the isolated towers were assumed to be 0.2592 Hz, 0.2864 Hz and 0.3404 Hz for the *x*-direction sway motion, *y*-direction sway motion and torsional motion, respectively. (i.e., the lower value of each modal pair).

4.2. Modal response

The dynamic correlation and phase of wind-induced response on the twin towers were investigated by examining each modal response. Fig. 3 shows that the *x*-direction motion is mainly contributed to by Mode 1 and Mode 4, the *y*-direciton motion is mainly contributed to by Mode 2 and Mode 3, and the torsional motion is mainly contributed to by Mode 5 and Mode 6. The total responses, such as the accelerations at the top of the building, were determined by Eq. (5). To reveal the relative importance of each mode, a normalized acceleration was calculated. The normalized acceleration is a ratio of the acceleration contributed by an individual mode to the total acceleration. The results are given in Fig. 4.

Fig. 4 shows that the same-direction and the opposing-direction sway motions tend to occur in alternative wind directions. When wind blows along the *x*-direction (i.e., around 100^o and 280^o), the motion of the twin towers will be mainly in opposing directions. When wind blows along the *y*-direction (i.e., around 10^o and 190^o), the motion of the towers becomes in the same direciton. From a structural design point of view, the opposite-phase motion has important consequences. The motion of Mode 3 may create large torsional moments on the foundation and the motion of Mode 4 may cause large stress on the structural links. The presented results indicate that in terms of magnitude, the opposing motions could be higher than the in-phase motions.

Fig. 5 further verifies the significance of the opposite-phase motion. Under the given condition

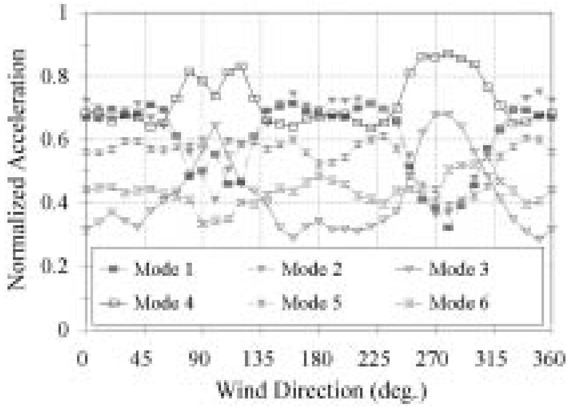

Fig. 4 Normalized accelerations of each mode

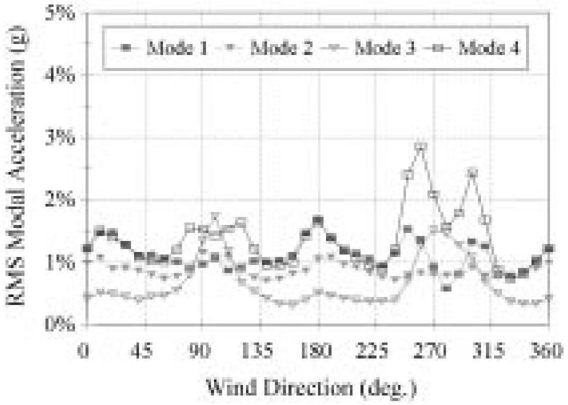

Fig. 5 Root-mean-square modal accelerations

that the wind speeds for all directions are equal to a 50-year return period value, Fig. 5 reveals the sensitive directions of wind-induced motions for each mode. Two maximum peaks occur for Mode 4 at 260° and 300°. These are the two direcitons where winds sweep over Tower A and induce large wake buffeting on Towers B and C. In these conditions, the opposite-phase motion becomes particularly significant.

4.3. Impacts of structural links on wind response

Structural links tend to make the two buildings experience the same level of motion (such as acceleration) by equalizing their energy. Fig. 6 shows the root-mean-square (RMS) acceleration in the *x*-direciton at the worst wind direction of 260° as a function of reference wind speed at gradient height. As a comparison, the estimated accelerations of two towers with no structural links between them are also plotted on the same figure. The results show that the links reduce the accelerations on Tower B, the tower with higher response, but increase the accelerations on Tower C, the tower with lower response. The extent rate of this reduction or increase is not a constant, ranging from 15% to

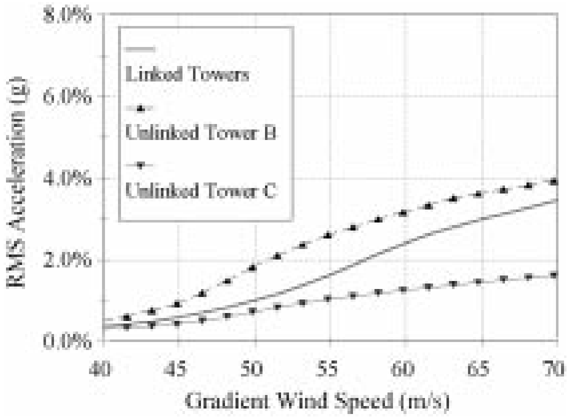

Fig. 6 *x*-direciton acceleration at 260°

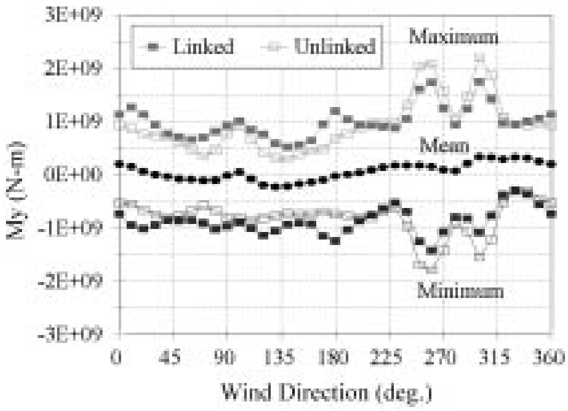

Fig. 7 Overturning moment M_y on Tower B

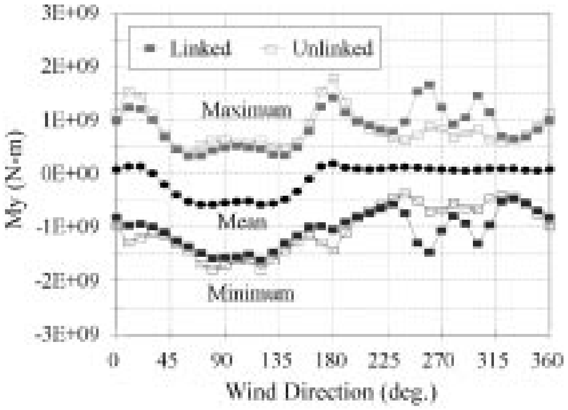

Fig. 8 Overturning moment of M_y on Tower C

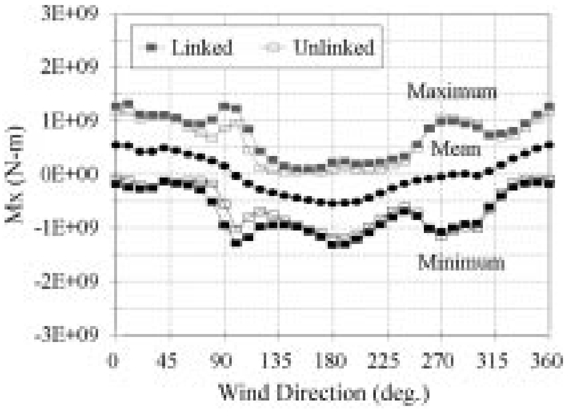

Fig. 9 Overturning moment of M_x on Tower B

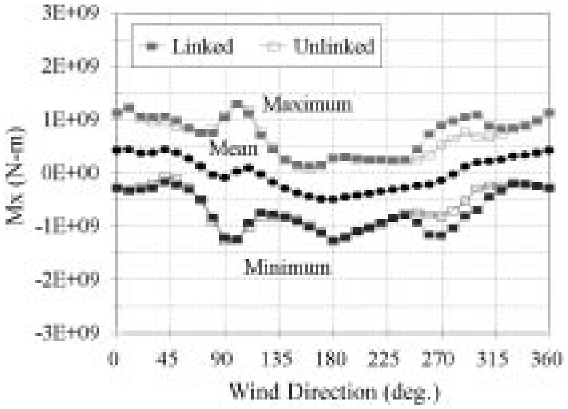

Fig. 10 Overturning moment M_x on Tower C

40%. This is because many factors may affect the impact of structural links on the dynamic response, such as the phase of wind buffeting on each tower.

Figs. 7 and 8 show the effects of the structural links on the overturning moments M_y at the podium level of each tower for various wind directions at the same wind speed (50-year return period). In terms of worst case loads regardless of wind direction, the structural links cause a reduction in overall peak loads by a factor of 0.79 for Tower B and a factor of 0.93 on Tower C.

Figs. 9 and 10 give similar plots for M_x. It is disappointing to note that instead of reduction, the links make both towers experience slightly higher loads. At the worst direction of 100^o, the reduction on Tower C is negligible, but the increase on Tower B is more than 20%, The reason for this unfavourable effect of structural links was further studied and will be explained in details in the following seciton.

The correlaiton of wind loads on each tower was examined using correlation factors, defined as follows :

$$\text{Correlation Factor} = \frac{P_{overall}}{[P_B] + [P_C]}$$

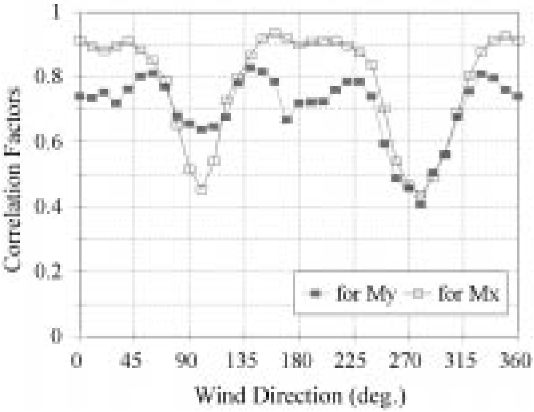

Fig. 11 Correlation factors of base overturning moments

where $P_{overall}$ = peak loads on the overall structure; P_B = peak loads on Tower B; and P_C = peak loads on Tower C. Note that for practical purposes, the peak loads include mean components which are fully correlated. Fig. 11 indicates that the correlation factors for base overturning moments are typically between 0.4 and 0.9. Referring to Fig. 4, it can be seen that the correlation factors take lower values when opposite motions are dominant. The main contributions to the overall torsional moments are from the difference of horizontal loads on each tower, and therefore the overall torsional moments are much higher than the summation of torsional loads on each individual tower.

4.4. Effect of frequency ratios

It was found that the frequency ratio between the pair modes was an important parameter for structurally linked towers. We examined the relations between the acceleration ratio and the frequency ratio. The acceleration ratio was defined as the ratio between the considered acceleration (i.e., the acceleration of linked towers) and the maximum acceleration of a corresponding unlinked tower, either on Tower B or Tower C. The natural fregency of each unliked tower was the same in the *x*-direction as mode 1 of the linked complex, and the same as mode 2 in the *y* direction. The frequency ratio was adjusted by changing the frequency of Mode 4 for *x*-direction motion and Mode 3 for *y*-direction motion. Figs. 12 and 13 present the acceleration ratios as a function of frequency ratios at wind directions 260^o for the *x*-direction motion and 100^o for the *y*-direction motion. Fig. 12 shows that, the *x*-direction acceleration on Tower C, if it is not linked with Tower B, will be smaller than that on Tower B, if it is not linked either, by a factor of 0.39. If the two towers are linked with a frequency ratio of 1.1, these two towers will experience the same level of acceleration with a magnitude smaller than that on the unlinked Tower B by a factor of 0.81. With decreasing the frequency ratio, the benefits of structural links tend to be reduced. When the frequency ratio reaches 1.0, both towers will experience the same level of higher accelerations.

Fig. 13 is a similar plot for the *y*-direciton acceleration at a wind direciton of 100^o. At this direction, Tower C will experience a higher acceleration than that on Tower B if they are not linked with each other.

Figs. 12 and 13 suggest that although structural links usually have positive impacts on dynamic response by improving the worst case, it is also possible that both towers' responses will be boosted to be the worst case, if the frequency ratio is near 1.0. Bearing in mind that in strong winds the

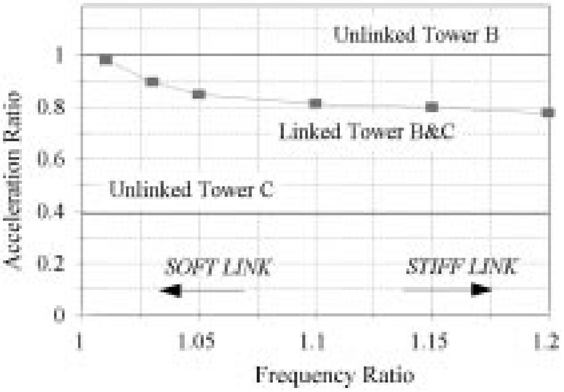

Fig. 12 *x*-direction acceleration ratio of 260°

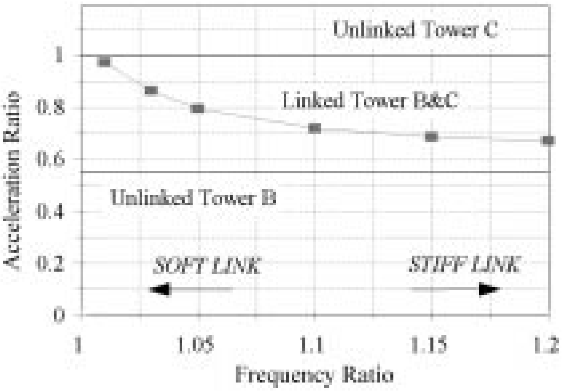

Fig. 13 *y*-direction acceleration ratio at 100°

building accelerations are mainly contributed to by resonance components, this phenomenon can be understood by a simple hypothetical case. Assume that between Tower B and Tower C, only Tower C is fully exposed to wind excitation and Tower B is sheltered. If these two towers are unlinked, the peak acceleration on Tower B will be zero and the peak acceleration on Tower C will be at a certain magnitude, say *a*. If these two towers are linked, each of the pair modes will contribute an acceleration of (*a*/2) as the generalized force remains the same but the generalized mass is doubled by including Tower B. If the frequency ratio of the pair modes is quite high and thus the cross-modal correlation is very small (i.e., in Eq. (5), $\rho_{ij} \rightarrow 0$), the total peak acceleration will be about 0.7*a* by combining the modal accelerations. However, with the frequency ratio approaching 1.0, the cross-modal correlation with be increased (i.e., in Eq. (5), $\rho_{ij} \rightarrow 1$) and therefore the peak acceleration will be about *a* associated with "beating" between the two towers.

For the studied twin towers, the frequency ratio is 1.14 for *x*-direction motion (Mode 1 and Mode 4) and only 1.02 for *y*-direciton (Mode 2 and Mode 3). Therefore, while the structural links show a positive impact on the *x*-direction response, the *y*-direction response becomes worse, as shown on Fig. 7 through 10.

5. Conclusions

1. Structural links may have significant effects on wind-induced dynamic response, even for weak connections. For the particular twin towers examined in this paper, the accelerations were changed by 15% to 40% and the overall peak wind loads were changed by 7% to 20%.
2. For a twin tower structure, the motions in opposing directions of the towers can higher than motions in same direction. Wake buffeting due to surroundings may further enhance the importance of opposite-direction motion. The correlation factors on peak loads are typically between 0.4 and 0.9 for the studied twin towers.
3. The frequency ratio of pair modes is and important parameter for wind-induced response of twin tower structures. If this tatio is well above 1.0, the cross-modal correlation becomes negligible and the worst case response tends to be reduced by structural links. In this case, the structural links are considered to have positive impacts on wind-induced response. However, if the frequency ratio is near 1.0, the cross-modal correlation becomes significant and therefore

both towers may experience the worst case response. In this case, the structural links have an adverse effect on wind-induced response.

4. The newly developed multi-force-balance system (MFB) provides a practical method for studying structurally linked tower structures. With this system the wind-induced responses on multi-towers could be predicted more precisely and more comprehensively.

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