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# Analysis of Wind Vibration Characteristics of a Narrow-shaped Steel Residential High-rise Building

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## ABSTRACT

A narrow-shaped high-rise steel residential building with a width-depth ratio reaching 6 around and a height-depth ratio larger than 6 is considered in this paper. Multi-point surface pressure measurement and high-frequency force balance (HFFB) wind tunnel tests were carried out for studying the wind-induced dynamic characteristics of the building. The dynamic responses especially the accelerations which are closely related to the serviceability performance of the building were determined and examined both in time-domain and frequency-domain. The spectral properties of the time-history results of dynamic responses have been compared with those of the frequency-domain results. The analyses and comparisons indicate that for flexural narrow-shaped high-rise residential buildings, the acceleration responses come to be more remarkable when the along-wind or across-wind directions roughly coincide with the shorter length direction. With the narrow-shaped cross-section reducing the vortex shedding in the across-wind direction, the peak accelerations are usually dominated by the along-wind direction. The torsional accelerations could be close to those of the along-wind direction, which seems to be affected by not only the fundamental mode but also the high order modes.

## 1. INTRODUCTION

High-rise residential buildings are often designed to narrow-shaped tall blocks with small concave-convex variations on the two longer sides in order to gain better daylight and ventilation performance. The wind vibration especially across-wind vibration characteristics of such kind of buildings are generally more different with a majority of office buildings containing roughly equal lengths and widths in plan.

With the advances of structural analysis and design for wind in the past decades, the along-wind dynamic responses of a high-rise building can be directly determined using the quasi-steady assumption and the concepts of orthonormal and influence functions developed by Davenport (1961, 1995). However, the quasi-steady assumption is not applicable to across-wind dynamic responses, resulting in many

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difficulties and remained problems in across-wind vibration investigations. In the area of wind vibration of high-rise buildings, most investigations were concerned on the commonly used office buildings with regular plan shapes, and those for narrow-shaped residential buildings were indeed seldom. The recommendations for loads on buildings proposed by Architectural Institute of Japan (AIJ 2004) presented some detailed regulations for determination of across-wind loads. Nevertheless they were only applicable to buildings with the width-depth ratio less than 5.

A commonly used procedure for determination of the across-wind dynamic responses of a high-rise building is to perform time- or frequency-domain analysis based on its wind tunnel experimental measurement. Quan (2002) conducted a series of high-frequency force-balance (HFFB) tests and measured the across-wind aerodynamic force spectral for tall buildings with typical cross sections. They also examined some related affecting factors on the aerodynamic force spectral. In this HFFB data-based analysis, the linearity assumption of fundamental mode shape was applied to estimate the first-order generalized aerodynamic force. In order to obtain a more theoretically accurate first-order aerodynamic force, or to find the high-order aerodynamic forces, multi-point surface pressure wind tunnel measurements are required (Gu 2006, Rosa 2012). Based on the measurement data, both the time-domain and the frequency-domain analyses can be carried out and compar3d with each other.

In this paper, both the multi-point surface pressure measurement and the HFFB tests were conducted for investigation of a narrow-shaped high-rise steel residential building with small concave-convex variations on the two longer sides. Frequency-domain analysis was mainly carried out and the results were compared with those of the time-domain analysis. The across-wind vibration characteristics of the building were summarized and the possible errors from the tests and computations were then discussed.

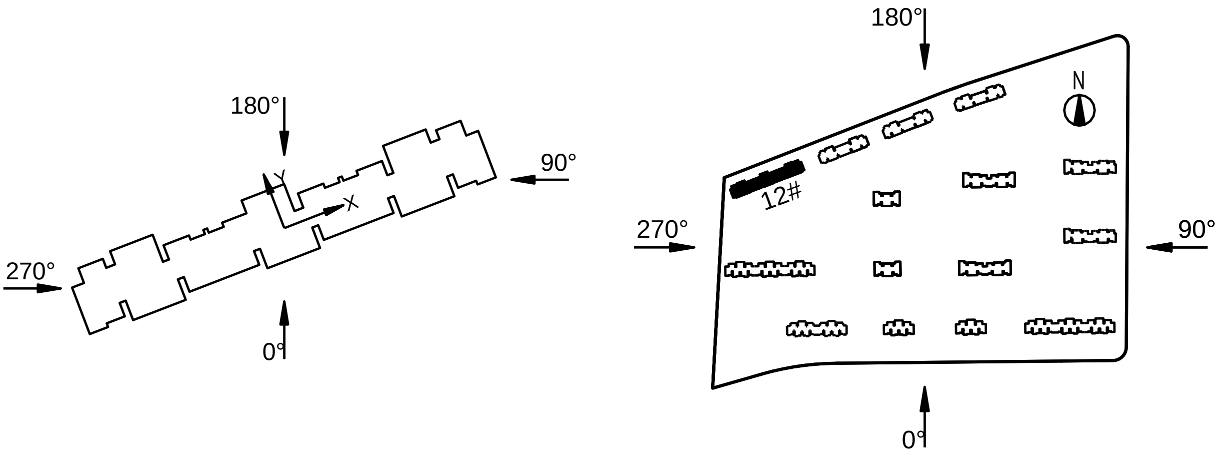
## 2. WIND TUNNEL TESTS

### *2.1 Brief Introduction of Wind Tunnel Tests*

Both the multi-point surface pressure measurement and the HFFB tests were performed at the ZD-1 closed-circuit boundary layer wind tunnel laboratory located in Zhejiang University, Hangzhou, China. The test section of the tunnel has an 18m length, 4m width and 3m height, providing a maximum wind speed of 55m/s.

According to the load code of China, the terrain category of the test building is type B, with a reference wind pressure 0.45kN/m$^2$ in a 50-year return period, and a reference pressure 0.30kN/m$^2$ in 10-year return period. The building in study is 15.7m$\times$94.8m in plan and 104.5m in height. The wind tunnel test model of the building was in a scale ratio of 1: 250. The inflow wind mean velocity was characterized by a power law with an exponent $\alpha$=0.16, while the turbulence intensity was set to 16.75% at 30m of elevation.

Since the height of each building in the building group (Fig. 1b) was similar, the shielding effect of the surrounding buildings on the considered one would be significant. Therefore two work conditions of wind tunnel tests, one for single building ,and the other for the same building but with all surrounding buildings in the group existing, were carried out. In the tests, total 24 wind attack angles with a step of 15° were considered. The definitions of wind attack angle and coordinate system are shown in Fig. 1.




(a) Plan of single building (b) Building group

Fig. 1 Definitions of wind attack angles and coordinate system

*2.2 Structural Properties and Computational Model*

The building structure in consideration is a steel framed structure with 37 storeys including 3 underground ones. The damping ratio if the structure was set to 0.02. A simplified multi-degree-of-freedom rigid-floor piece model was established in the current wind-induced response computations. Each structural storey was described in two translational degree-of-freedoms and one torsional degree-of-freedom, with 111 degree-of-freedoms in total.

The natural frequencies determined by the software SATWE which is widely used in building design institutes of China and the current simplified model are listed in Table. 1. It is obvious that the simplified model gives a much good agreement with SATWE.

Table. 1 Comparison of natural frequencies

| Mode | First mode | Second mode | Third mode |
|---|---|---|---|
| Simplified | 0.260 | 0.299 | 0.367 |
| SATWE | 0.269 | 0.300 | 0.345 |

*2.3 Approaches for Determination of Wind Induced Responses*

Three approaches have been used in this paper to find the across-wind responses. The first is time history analysis approach, based on the data of the synchronous multipressure sensing system (SMPSS) technique. The second is high-frequency pressure integration (HFPI) technique, analyzing the SMPSS test data in the frequency-domain. The last one is the frequency-domain analysis based on the HFFB test data. The three approaches are all on the basis of the random vibration theory (Simiu 1986).

There are some evident distinctions among the above three approaches:

The time history analysis uses the Newmark-$\beta$ method in equation solution, dealing with the data in time-domain and both the fundamental and high-order mode shapes being considered, while the HFPI technique handles the data in frequency-domain, taking only the fundamental mode into consideration. The approach for getting is different between the method and the method. The HFPI approach tries to find the generalized aerodynamic forces in strict accordance with the definition, while the HFFB technique has to assume the forces proportional to the base moment.

In this paper, we focus the dynamic analysis on the acceleration responses.

## 3. ACCURACY OF COMPUTATION IN FREQUENCY-DOMAIN

The results of acceleration at different wind attack angles, computed by the three dynamic analysis approaches: time history analysis, the HFPI and HFFB techniques, have been shown in Fig. 2 to Fig. 6 in which the torsional acceleration is provided in the way of a linear acceleration at the building corner for convenience of comparison with the translational acceleration.

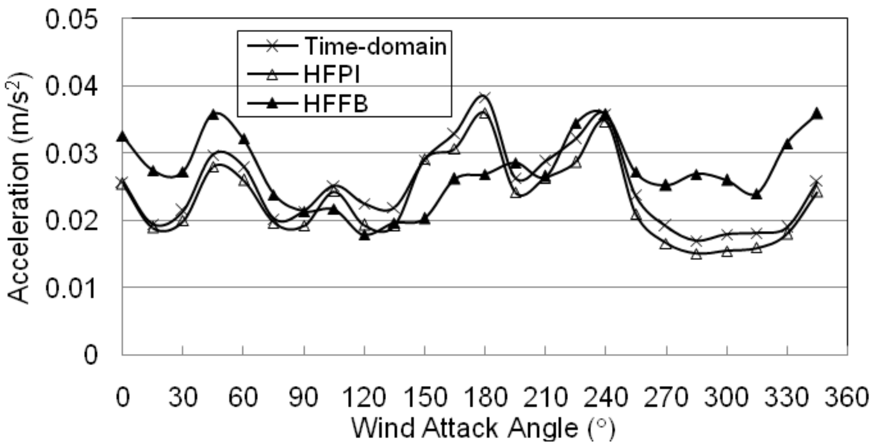



Fig. 2 Acceleration along *x*-direction

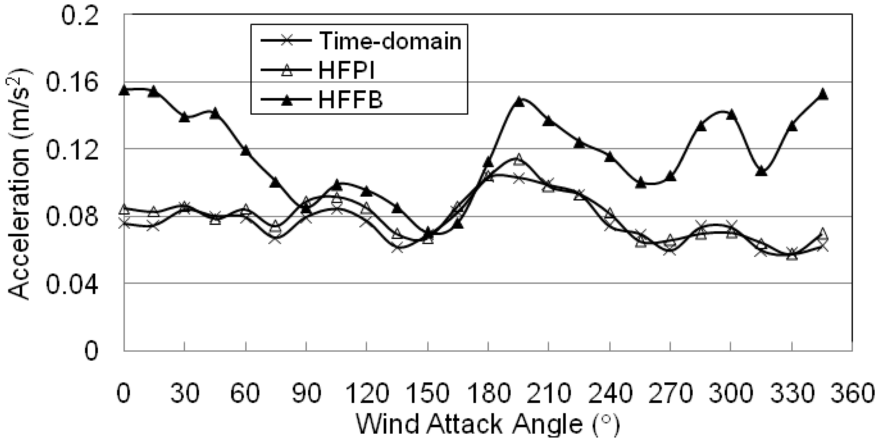



Fig. 3 Acceleration along *y*-direction

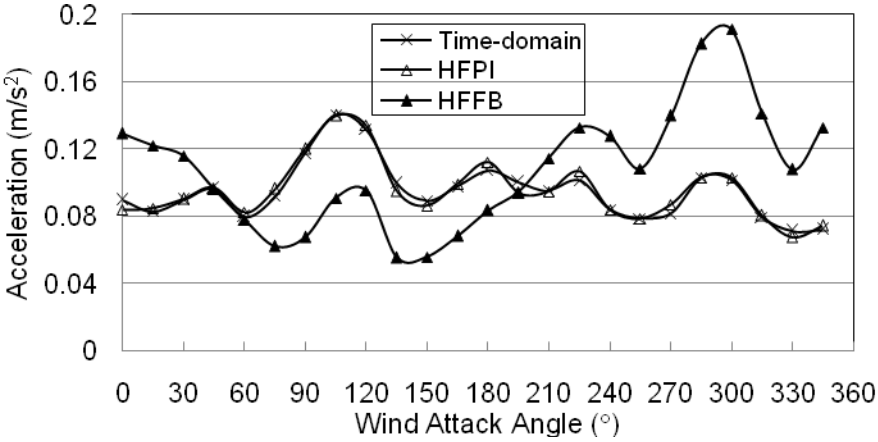



Fig. 4 Torsional acceleration responses

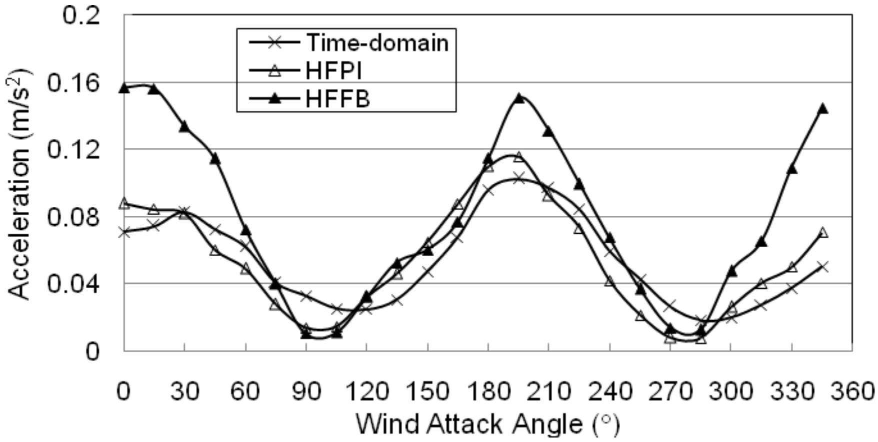



Fig. 5 Along-wind acceleration responses

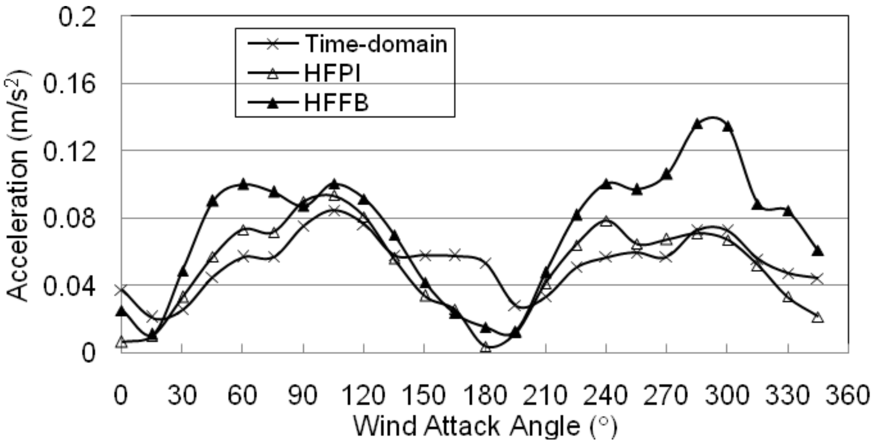



Fig. 6 Aross-wind acceleration responses

Most of the results determined in time-domain and in frequency-domain are similar. However, for the wind attack angle of 180°, the across-wind accelerations obtained in different approaches get separated. As for the reason, in the process of time history analysis, it resolves the accelerations into along-wind and across-wind first, and gains the mean square root of them. On the other hand, in the process of frequency-domain analysis, it finds the mean square root first, and then resolves into along-wind and across-wind directions. In other words, using random vibration theory to solve the wind induced responses will remove the direction of motion.

The results of HFFB show more deviation than HFPI in the comparison, especially in the accelerations along *y*-axis, whose stiffness is smaller. At certain points the deviation becomes as big as the value itself. Whereas, the variation trends in along-wind and across-wind accelerations seem to be the same.

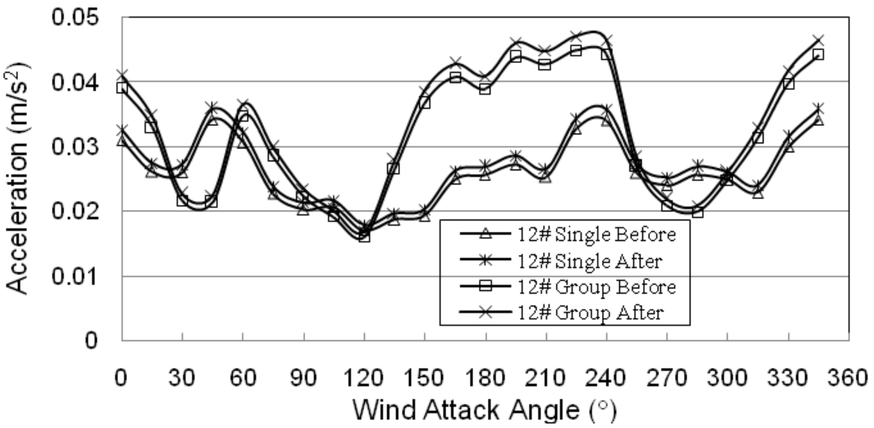
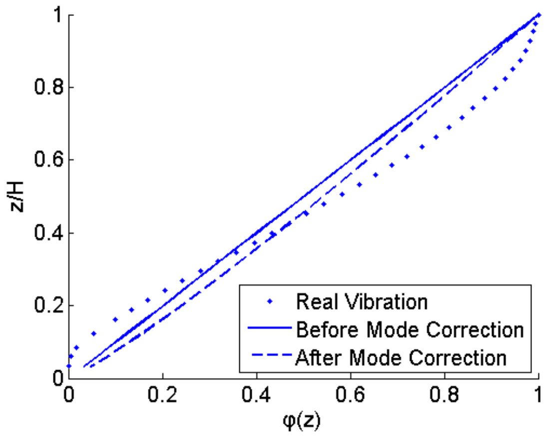
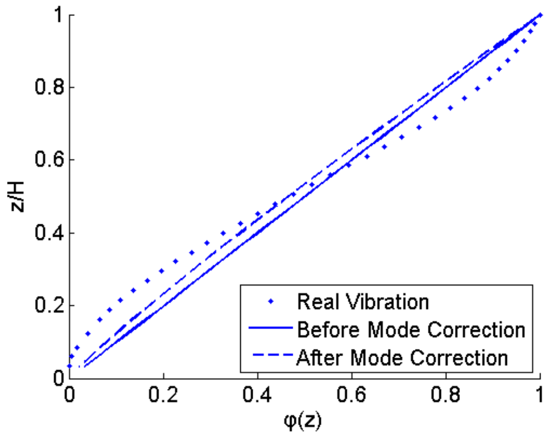



Fig. 7 Comparison of accelerations before and after mode shape correction

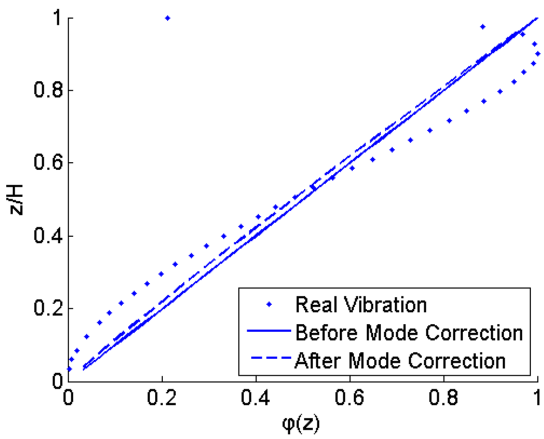
Unlike the HFPI approach, which find the generalized aerodynamic forces in strict accordance with its definition, the HFFB approach only has the time history of base bending moment and torque, hence it has to assume the fundamental mode being linear for using the base bending moment and torque to approximate the generalized aerodynamic force. Considering the influence from nonlinearity, several scholars suggested some mode shape correction techniques (Xu 1993). Fig. 7 shows the comparison of accelerations between before and after mode shape correction. It implies that the correction seems not so effective for improving the results of HFFB. Fig. 8 presents the contrast of mode shapes among real vibration, approximate vibration before and after correction. That is to say, it is not so satisfied for us to use the power function for mode shape approximation, which contributes to the HFFB result correction.




(a) Fundamental mode along *x*-direction



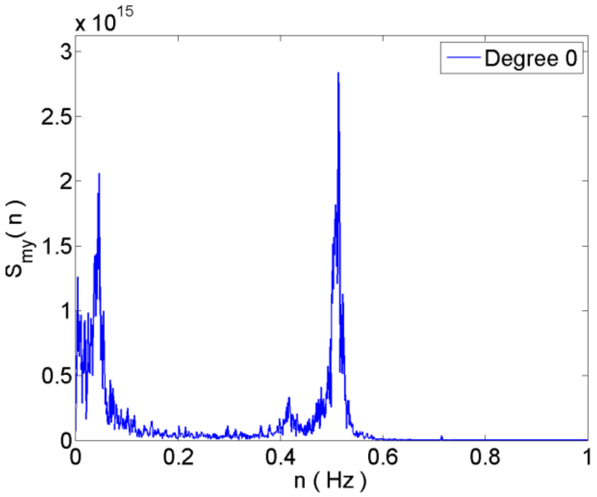

(b) Fundamental mode along *y*-direction



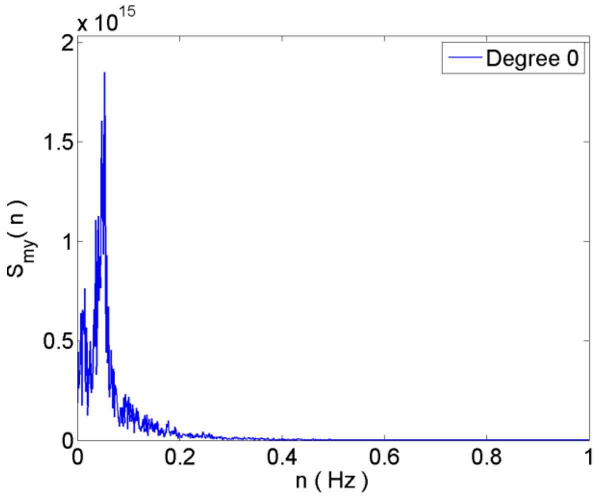

(c) Torsional fundamental mode

Fig.8 Comparison of mode shapes

On the other hand, the deviation of measurement for the base moment and torque in HFFB tests will result in obvious errors to dynamic responses. It can be seen from Fig. 9 that there is an extra peak at the high frequency area of the base moment power spectrum density (PSD) curve obtained form HFFB test. This abnormal result may be caused by the electronic or mechanical errors in the test. Further investigations are much required.




(a) HFFB




(b) HFPI

Fig.9 PSD of base moment at 0° wind attack angle

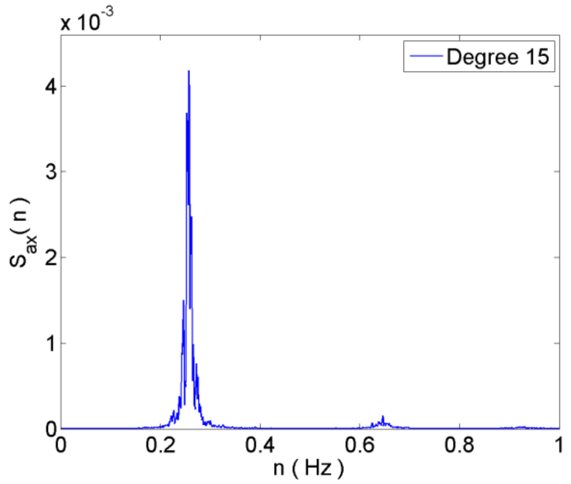
## 4. WIND INDUCED DYNAMIC PROPERTIES

*4.1 Dynamic Properties of the Structure*

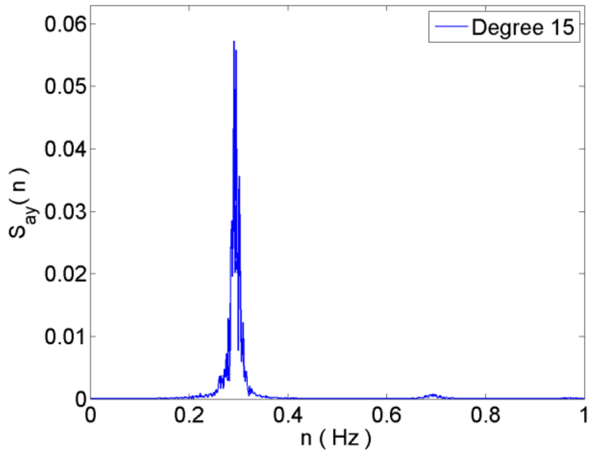
Fig. 5 indicates that the two extreme points of along-wind acceleration appear at 15° and 195° wind attack angle, respectively. That is to say the along-wind directions correspond to the weaker axes directions. From Fig. 6 it is seen that the two across-wind acceleration extremes appear at 105° and 285° respectively, implying that in this case the across-wind direction becomes the weaker axis direction.

It is worth mentioning that although the building has a much different width and depth, the across-wind acceleration does not exceed the along-wind acceleration.

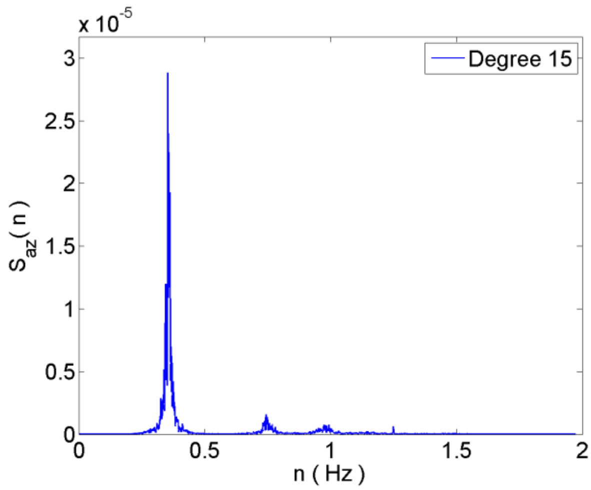
Fig. 10 presents the spectra of the *x*- and *y*-axis and torsional accelerations at wind attack angle of 15°. It is seen that the acceleration responses are controlled by the fundamental modes, whose natural frequencies are 0.260Hz, 0.299Hz, 0.367Hz, respectively. The effect of other high-rise buildings nearby produces seldom effect on this dynamic property in this test, only changing the accelerations in magnitude. In addition, the high order modes take part in the torsional acceleration, whose contributions can be almost ignored in the translational accelerations.




(a) Along *x*-axis




(b) Along *y*-axis

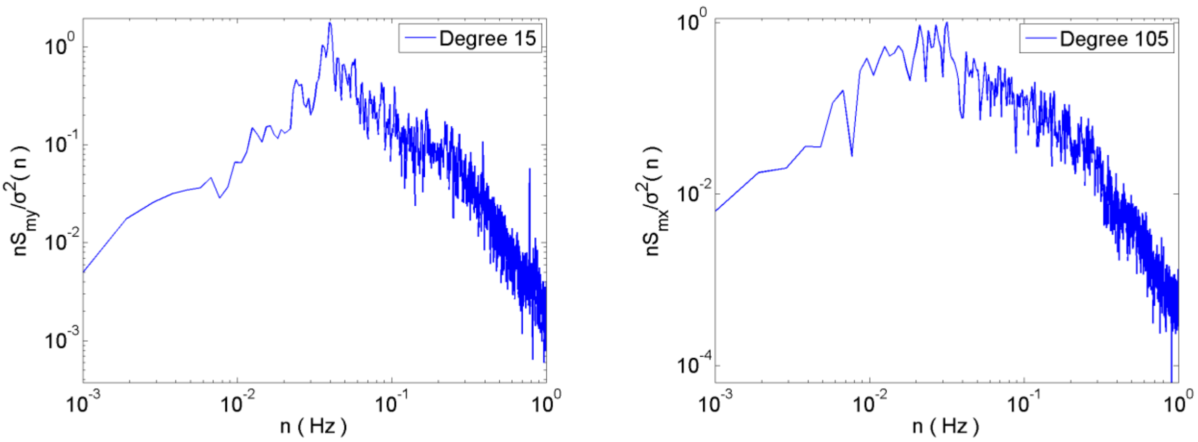



(c) Torsional direciton

Fig.10 PSD of acceleration at 15° wind attack angle

*4.2 Wind Turbulence Properties*

The wind induced responses are mainly caused by the atmospheric turbulence, which exists at each wind attack angle. It can be seen in Fig. 11 that the vortex shedding comes obvious in the across-wind direction, while the windward side is the longer one. However, when the windward side is the shorter one, the side faces in the along-wind direction of the building are so long that obstruct the formation of vortex shedding, resulting in the reduction of wake excited responses. Finally, the along-wind acceleration exceeds the across-wind one, although the building shows such a difference along the *x* and *y* directions both in size and stiffness.




(a) Windward side longer (b) Windward side shorter

Fig. 11 Normalized PSD of across-wind base bending moment

## 3. CONCLUSIONS

Unlike the HFPI method and the time history analysis, results from the current HFFB test had obvious deviation. Nonetheless the three dynamic analysis methods display the same trend in the acceleration variation with the wind attack angles.

With regard to the narrow-shaped high-rise building, whose sizes and stiffness along the width and depth directions are much different from each other, the acceleration responses come to be more remarkable when the along-wind or across-wind directions roughly coincide with the shorter length or weaker stiffness direction. The wake excited responses caused by vortex shedding are reduced by the so long side faces, resulting in the maximum across-wind acceleration being less than the along-wind direction. The wind attack angles have small impact on the torsional acceleration, while the high order modes display an obvious influence on it.

## REFERENCES

Architectural Institute of Japan AIJ. (2004), *AIJ 2004 Recommendations for loads on building*, Tokyo, Architectural Institute of Japan.
Davenport. A.G, M.A, and M. A.Sc. (1961), "The application of statistical concepts to the wind loading of structures", *Proc Inst Civil Eng*, **19**(a), 449-472.
Davenport. A.G. (1995), "How can we simplify and generalize wind loads?", *J. Wind Eng. Ind. Aerodyn.*, **54**, 657-669.
Gu. M, and Ye. F. (2006), "Frequence domain characteristics of wind loads on typical super-tall buildings", *Journal of Building Structures*, **27**(1), 30-36.
Quan. Y, and Gu. M. (2002), "Power Spectra of Across-wind Loads on Super High-rise Buildings", *Journal of Tongji University,* **30**(5), 627-632.

Rosa. L, Tomasini. G, Zasso. A, et al. (2012), “Wind-induced dynamics and loads in a prismatic slender building: A modal approach based on unsteady pressure measurements”, *J. Wind Eng. Ind. Aerodyn.*, **107**, 118-130.

Simiu. E, and Scanlan. R.H. (1986), *Wind effects on structures: an introduction to wind engineering*. John Wiley.

Xu. Y.L, and Kwok. K.C.S. (1993), “Mode shape correction for wind tunnel tests of tall buildings”, *Engineering Strctures*, **15**, 387-392.