WIND-INDUCED RESPONSE CONTROL MODEL FOR HIGH-RISE BUILDINGS BASED ON RESIZING METHOD

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Abstract. A variety of methods have been applied to reduce the effect of the wind-induced vibration of a high-rise building as the excessive wind-induced vibration at the top of a high-rise building can cause physical and psychological discomfort to the user or the residents. For structural engineers, the most effective approach to control the wind-induced responses of high-rise buildings would be to control the stiffness or natural frequency of the building. This paper presents a practical design model to control the wind-induced responses of a high-rise building. In the model, the stiffness of a high-rise building is maximized to increase the natural frequency of the building by the resizing algorithm. The proposed design model is applied to control the wind-induced vibration of an actual 37-storey building during the initial stage of its structural design.

Keywords: high-rise building, serviceability design, wind-induced vibration, resizing algorithm, structural optimization.

Introduction

Buildings can be vibrated by earthquakes and wind. The duration of earthquake-induced vibration is relatively short and related to the safety of building structures such as the risk of collapse. To reduce the earthquake damage, methods that dissipate the earthquake energy transmitted to the buildings using the dampers or that reduce the earthquake energy applied to the buildings using the isolators are used (Lee *et al.* 2009; Marshall, Charney 2012; Sayani *et al.* 2011; Taflanidis, Jia 2011). On the other hand, the duration of wind-induced vibration is relatively long compared to that of earthquake-induced vibration and is related to the serviceability of building structures (Kijewski, Pirnia 2007).

Generally, in high-rise buildings, the wind load rather than the earthquake load acts as a critical design condition (Kim, Kanda 2008). Recently, due to the development of construction technology and high-strength materials, the height of high-rise buildings has increased and the weight has lessened. These factors make high-rise buildings sensitive to wind. Therefore, it is important to control the wind-induced vibration of high-rise buildings. To reduce the effect of wind-induced vibration, a variety of methods have been applied to the structural design of these buildings. These methods include an aerodynamic design of the buildings' shapes, control of the damping ratios, and the application of an effective structural system (Kareem *et al.* 1999).

An aerodynamic design pertaining to the shape of a building seeks to reduce the wind-induced responses of high-rise buildings by changing the shape of the building. Examples of the common aerodynamic design include the modification of the corner geometry of a building, the addition of openings to a building, and the control of the building aspect ratio. The second method generally employed to reduce the wind-induced responses of high-rise buildings is associated with the use of additional damping devices such as viscoelasticity dampers, fluid dampers, tuned liquid dampers, and tuned mass dampers (Lewandowski, Grzymislawska 2009; Patil, Jangid 2011). For structural engineers, both of the aforementioned methods are not always considered as a practical and feasible solution, since the decision to change the shape of a building and to use additional damping devices cannot be made by the structural engineer alone but instead require the consent of the owner and the architect.

Therefore, for structural engineers at the initial phase of the structural design, the most effective approach to control the wind-induced responses of high-rise buildings would be to control the stiffness or natural

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frequency of the building. It is known that the level of the wind-induced vibration of a high-rise building with a natural frequency range of 0.1~1.0 Hz can be assumed to be inversely proportional to the natural frequency of the building (Tallin, Ellingwood 1984; Griffis 1993). Tallin and Ellingwood (1984), Chan and Chui (2006), and Chan *et al.* (2009) have proven that the wind-induced response can be reduced by increasing the natural frequency of a building.

As such, it would be necessary for structural engineers to have a practical method to increase the stiffness or natural frequency of a high-rise building without a significant increase in the structural weight or construction cost. Traditional trial-and-error based structural design methods may not be practical and efficient for structural engineers since a high-rise building is composed of numerous structural members to be designed and designers do not have enough information about which members are active to the natural frequency. The information about the active members may be obtained through a complicated process to solve the functional formula of the mass and stiffness of all structural members in a high-rise building, as the natural frequency is determined through a combination of the mass and stiffness of all structural members.

Research on design methods for the wind-induced vibration of high-rise buildings has been actively carried out. Chan and Chui (2006), Chan *et al.* (2009, 2010) proposed optimal design methods to minimize the structural weight or cost while satisfying the serviceability condition using cross sections of members as a design variable. They used the Optimality Criteria (OC) as an optimization tool. OC has a weakness in that it modifies the cross sections of members using a recursive resizing algorithm until the optimality criteria are satisfied while also requiring a sensitivity analysis. Li *et al.* (2011) presented an optimal wind design method using a micro-GA. This approach minimizes the structural cost subjected to the constraints on the displacement and acceleration. This requires a recursive analysis and an excessive amount of computation time. Petrini and Ciampoli (2012) presented a probabilistic framework for the performance based wind design of high-rise buildings. It probabilistically evaluates the relationship between the velocity of wind and the performance of the high-rise building. It requires an iterative analysis to calculate the probability and is difficult to apply to practical work.

In this paper, a practical design method based on the resizing algorithm, which does not require an iterative structural analysis, is presented to control the wind-induced responses of a high-rise building. In the resizing algorithm, active members, which have a relatively large influence on the magnitude of the lateral stiffness or the natural frequency to be controlled, are selected and the cross sections of the active members are resized. The proposed design method is applied to control the wind-induced vibration of an actual 37-storey building during the initial stage of its structural design.

1. Evaluation of the wind vibration

The wind-induced vibration of a high-rise building is evaluated mostly in terms of the root-mean-square (RMS) acceleration or the peak acceleration under extreme wind conditions as specified in various codes (ISO 6897 1984; NBCC 1995; ASCE 2006; AIJ 2004; Solari 1982; Boggs 1997; Cermak 2003). Due to the aerodynamic effects of the wind, a high-rise building vibrates not to the along-wind direction alone but also to the cross-wind and torsional directions. Consequently, to check the serviceability of a high-rise building under the action of wind loads, an evaluation of the wind-induced vibration needs to be performed on those directions.

There are two approaches for structural engineers to evaluate the wind-induced vibration of a building (Kijewski, Kareem 1998). In the first and more common approach, the wind-induced vibration may be estimated using simplified equations (NBCC 1995; ASCE 2006; AIJ 2004). Several codes and standards permit predictions of the wind-induced responses of a building using simplified equations during the preliminary design phases. However, there are limitations when seeking to predict the wind-induced responses precisely through simplified equations, as a high-rise building subjected to wind loads shows complex behavior due to the vortex effect and the interaction between the building and the wind.

In the practical structural design of a high-rise building with a relatively large building aspect ratio, more accurate methods of predicting the wind-induced responses are used. The most accurate mean of predicting the wind-induced responses of a high-rise building is to perform a wind-tunnel test during the initial design phase. Presently, one of the most commonly used evaluation methods for wind load and wind-induced responses in a wind-tunnel test is the High Frequency Force Balance (HFFB) method (Cermak 2003; Tschanz, Davenport 1983). In the HFFB method, the aerodynamic wind force spectrum is directly measured by the wind-tunnel test. The wind force spectrum measured by the wind-tunnel test, as shown in Figure 1, is used to estimate the wind-induced responses of a building analytically using the random vibration theory. The vertical axis in Figure 1 has a physical trend that is
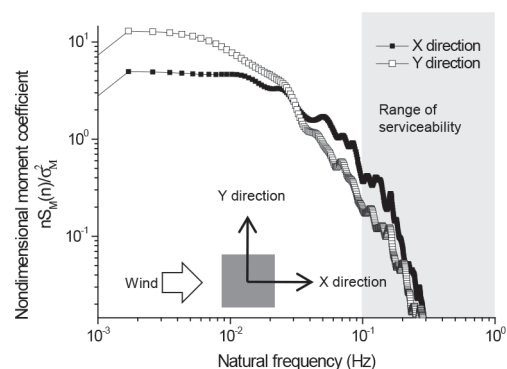

Fig. 1. Typical along-wind and across-wind aerodynamic base moment spectrum

proportional to the wind-induced vibration of a building, where n, $S_M(n)$, and σ_M are the natural frequency, the power spectral density and the standard deviation of the overturning moment, respectively. The spectrum in Figure 1, within the range of 0.1~1.0 Hz corresponding to the range of the natural frequencies of high-rise buildings, shows that the spectrum values decrease as the values of the natural frequency increase. This means that the wind-induced responses can be reduced by increasing the natural frequency of a high-rise building.

2. Resizing algorithm

The resizing algorithm is used to increase the natural frequency of a high-rise building by improving the stiffness of the building (Park, H. S., Park, C. L. 1997; Park *et al.* 2008; Seo *et al.* 2008). In the resizing algorithm, to increase the natural frequency of a high-rise building, the stiffness of the building structure is improved by reducing the lateral displacement at the top of the building without increasing the weight of the structure. Because the maximum values of displacement and acceleration, which are used as indexes for serviceability evaluations, occur at the top of the building, the resizing algorithm minimizes the displacement of the top of the building. Generally, the maximum acceleration of buildings is proportional to the maximum displacement of buildings. The displacement participation factor defined by each member's contribution to the displacement to be reduced consists of six components: axial, bending (Y and Z local axes), shear (Y and Z local axes), and torsional displacement components. Based on the calculated member displacement participation factors, active members can be selected to reduce the displacement and the cross sections of the active members can be modified for effective control of the displacement. To reduce the displacement without increasing the weight of the structure, a certain portion of structural material in the non-active members may be taken out and added to the active members. The amount of material to be resized depends on the member displacement factors and is determined by resizing algorithms formulated in the form of an optimization problem.

In the formulation of the resizing module, the objective function is expressed in terms of the displacement to be reduced and minimized with the assumption that the displacement participation factor of each member is inversely proportional to the change in weight of the member:

$$Minimize \quad \sum_{i=1}^{m} \frac{\delta_i}{\beta_i} =$$

$$\sum_{i=1}^{m} \frac{1}{\beta_i} \int_0^{l_i} \left(\frac{F_X f_X}{E\,A} + \frac{F_Y f_Y}{GA_Y} + \frac{F_Z f_Z}{GA_Z} + \frac{M_X m_X}{GI_X} + \frac{M_Y m_Y}{EI_Y} + \frac{M_Z m_Z}{EI_Z} \right)_i dx; \quad (1)$$

$$Subject\ to \quad \gamma \sum_{i=1}^{m} \rho_i A_i l_i = \sum_{i=1}^{m} \rho_i (\beta_i A_i) l_i , \quad (2)$$

where: δ_i and β_i are the displacement participation factor and the weight modification factor of the i-th member, respectively. The cross section of the i-th member is resized according to β_i; m is the total number of members in a structure and l_i is the length of the ith member; E and G are the modulus of elasticity and the shear modulus of elasticity, respectively; A, A_Y, and A_Z are the axial and shear areas of the cross section; I_X, I_Y, and I_Z are the torsional and flexural moments of inertia for the cross section; F_X, F_Y, F_Z, M_X, M_Y, and M_Z are the member forces and moments due to the wind load; f_X, f_Y, f_Z, m_X, m_Y, and m_Z are the member forces and moments due to the unit load at the top of the structure; ρ_i is the weight density of the ith member. In Eqn (2), γ is the total weight control factor to control the total weight of the structure. If the value of γ is set to 1.0, the displacement at the top of the building is minimized on the condition that there is no change in the total weight of the structure before and after resizing. Using the weight control factor, the relationship between the natural frequency and weight for a high-rise building can be obtained. The relationship between the natural frequency and the total weight for a high-rise building may allow structural engineers to estimate adequate structural weights in order to obtain the maximum natural frequency.

Eqns (1) and (2) are transformed into the unconstrained minimization problem in Eqn (3) by introducing the Lagrange multiplier:

$$Minimize \quad \delta_t = \sum_{i=1}^{m} \frac{\delta_i}{\beta_i} + \lambda \left\{ \sum_{i=1}^{m} (\beta_i W_i - \gamma W_i) \right\}, \quad (3)$$

where: δ_i and λ are the transformed pseudo objective function and the Lagrange multiplier, respectively; W_i is $\rho_i A_i l_i$. Taking derivatives of the Eqn (3) with respect to β_i and λ and setting them to zero, the weight modification factor β_i of the i-th member is given in Eqn (4):

$$\beta_i = \gamma \sqrt{\frac{\delta_i}{W_i}} \frac{\sum_{i=1}^{m} W_i}{\sum_{i=1}^{m} \sqrt{\delta_i W_i}}. \quad (4)$$

If the value of the displacement participation factor of the k-th member is relatively larger than the values of other members, the value of the weight modification factor of the k-th member becomes greater than 1.0. Based on the values of the weight modification factors of the members, the resizing module in this paper increases the stiffness of a building by changing the cross sections of the members.

3. Application to control the wind-induced vibration of a high-rise building

3.1. Model for controlling the wind-induced responses of a high-rise building

By combining the structural analysis module, the wind-induced vibration evaluation method, and the resizing algorithm with the weight modification factors, the design model to control the wind-induced responses of a high-rise building proposed in this paper is shown in Figure 2. For a given preliminary design satisfying the strength requirements based on design specifications, the level of wind-induced vibration of a high-rise building is estimated. If the estimated wind-induced responses of the preliminary design do not satisfy the serviceability requirements, the cross sections of members are resized according to the modification factors in Eqn (4) in the steps 6–9 in Figure 2. At this step, although the calculated wind-induced responses satisfy the serviceability requirement, the cross sections of members may be resized to reduce the total weight of the structure by specifying the values of the weight control factor in Eqn (2). In the final step, the strength requirements of resized members are checked using the modified cross sections.

In this study, it is assumed that the member forces are constant despite the fact that the cross sections of members are changed by the proposed method. In the indeterminate structures, the member forces are changed according to the cross sections of members. However, in high-rise buildings, the changes of the member forces ac-

cording to the changes in the cross sections of the members are small (Baker 1990; Charney 1991). Therefore, it can be assumed that the member forces are constant in the resizing algorithm.

3.2. Actual 37-storey building structure

The proposed design method is applied to the control of the wind-induced vibration of a 37-storey residential complex building structure built in 2003. As shown in Figures 3 and 4, the height, width, and depth of the building are 127.2 m, 27.4 m and 28.2 m, respectively.

This example is located in a downtown area with the stiff soil (soil profile type S2, AIK 2000). The shear wall, outrigger, and flat plate are used as its structural system. As the lateral load resisting system, outriggers are
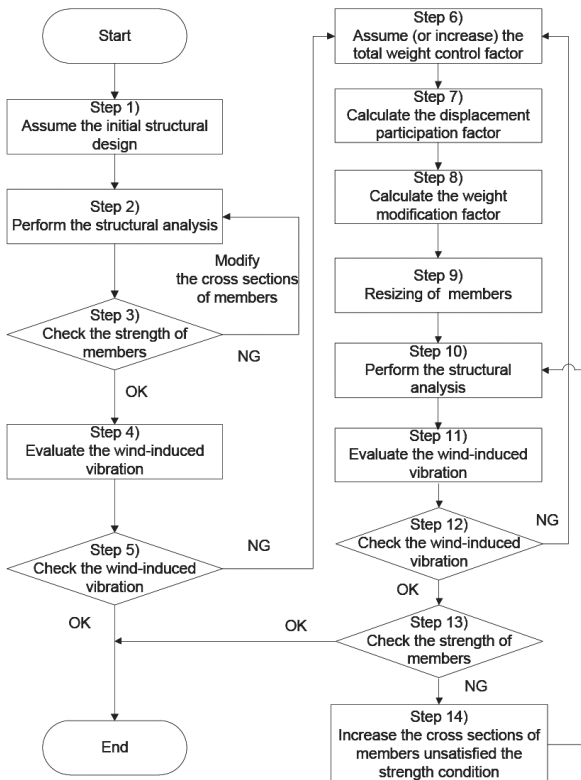

Fig. 3. Perspective view of the 37-storey building

Fig. 2. Design model to control the wind-induced responses of a high-rise building
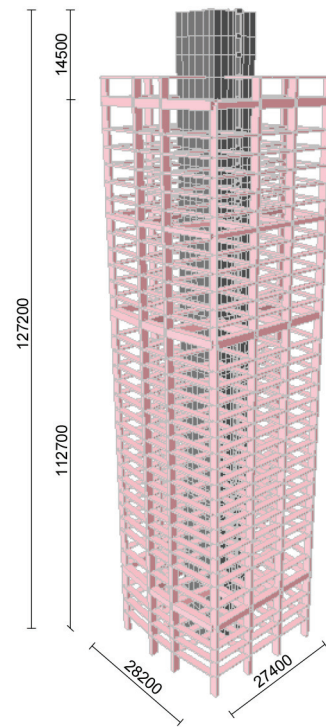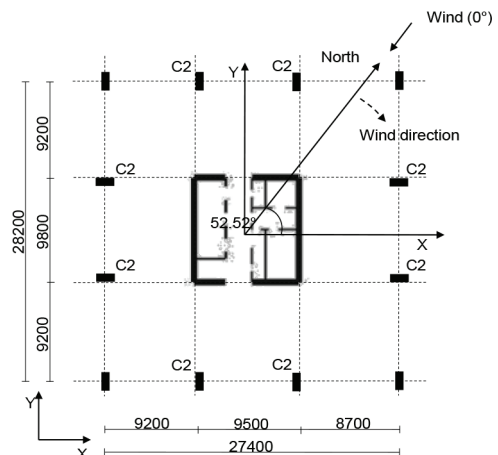

Fig. 4. Plan view of the 37-storey building

installed at 5, 22, and 34 storey levels. For structural analysis of the model, the Midas Gen software (MIDAS 2012) is used. The structural nodes at the supports located at the base are fixed in every direction. In the modelling, 1,986 different beam-type elements are used to model structural members for beams and columns. In the wall members, 1,978 wall-type elements are used. For this example, reinforced concrete with three different levels of the compressive strength of 27, 35, and 40 MPa are used. Every joint is modelled as a rigid joint. At every floor, the diaphragm effect is considered. The interaction between the ground and structure, the long-term loading effect, and the ground settlement of ground are not considered. In this study, the dead load, live load, earthquake load, and wind load are considered, and the earthquake load and wind load are calculated according to the AIK specifications (2000). To calculate the member forces and deformation of the structure, a linear static analysis is performed and KCI (2003) is used for the structural design.

The lateral load resisting structural system used in this example is the outrigger system. In the outrigger system, the key elements to resist the lateral load are shear wall, outrigger, and external columns. Therefore, in this study, the shear wall, the outrigger, and the columns are considered in the resizing algorithm, while flat plates as a vertical load resisting structural system are not considered in the resizing algorithm. The sections of flat plates do not change before and after the application of the resizing algorithm.

This example satisfies with the acceptance criteria of NBCC (1995), which are 10 and 30 mg for a resident and office building, respectively. Therefore, if the optimal model presented in section 3.1 is used, the resizing method would be not necessary in this example because the wind-induced vibration condition is satisfied. However, to evaluate the efficiency of this optimal model, the resizing method was applied only once, and then the results of the preliminary design and the final design obtained from the proposed model were compared.

In the process of the resizing, the total weight control factor γ is set to 1.0 so that the total structural weight is not changed before and after the application of the resizing algorithm. The displacement participation factors of the columns, beams, and shear walls are then computed and used for the calculation of the modification factors β_i for the structural members as defined in Eqn (4). The weights according to the members of the preliminary design (step 1), the design after resizing (step 9), and the final design after a strength check (step 14) are summarized in Table 1. The resizing method in this study uses continuous sections as a design variable. Therefore, the sections determined by the resizing method may be impractical sections and it may be necessary to modify these sections. Also, in this study, to prevent the thickness of shear wall from becoming impractical, a constraint condition stating that the thickness of the shear wall will change within 30% is additionally set. For these reasons, although the total weight control factor is set to 1.0, the weight of the design after resizing is reduced

to 0.94% compared to the weight of preliminary design. The weight of the design after the strength check is increased to 0.74% compared to the weight of the design after resizing and is decreased to 0.21% compared to the weight of the preliminary design. As shown in Table 1, the weight of the outrigger is moved to the weight of shear wall. The sections of column C2 in Figure 4 and the shear wall in the preliminary design and the design after the strength check along with the height are shown in Figures 5 and 6, respectively. The sections of the column and shear wall are increased at the lower level and are decreased at the upper level. Thus, the weight of the lower level increases and the weight of the upper level decreases, as shown in Figure 7. The maximum lateral displacements of the preliminary design and the design after the strength check are summarized in Table 2. The maximum displacements of 215.90 mm and 183.70 mm are decreased to 179.80 mm and 159.00 mm in the X and Y directions, respectively. The distribution of the lateral displacement of the building is shown in Figure 8.

In this paper, the resizing method is adopted to increase the natural frequency of the building for the control of the wind-induced response of the building. The natural frequencies of the building structure in the X and Y di-

Table 1. Weight change of the example according to the application of the resizing method (Unit: MN)

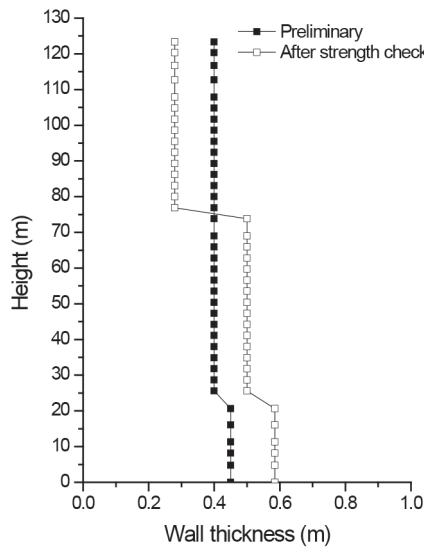|	Columns	Outriggers (beams)	Shear wall	Sum
Preliminary	46.25	81.45	78.95	206.66
After resizing	46.47	77.34	80.92	204.72
After strength check	47.98	77.34	80.92	206.23
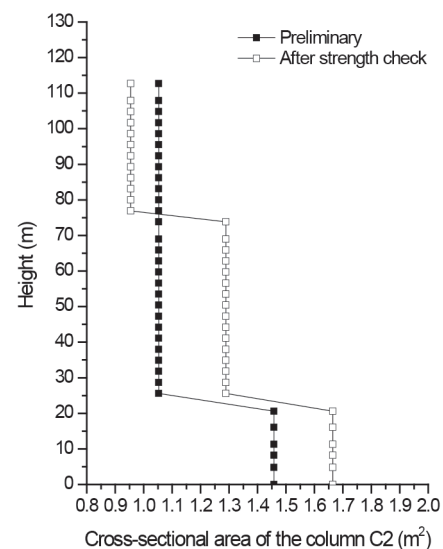
Fig. 5. Changes in the cross sections of column C2
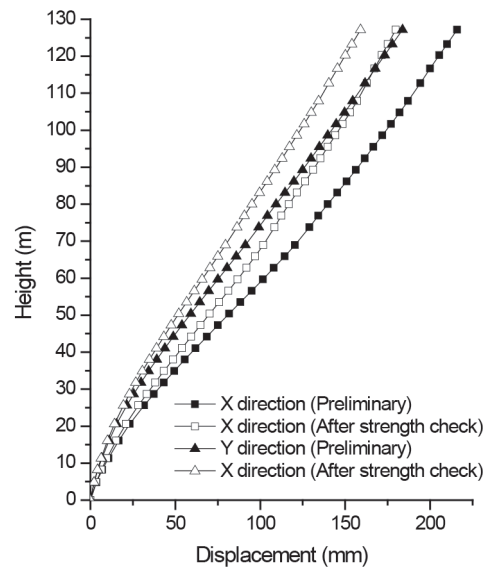
Fig. 6. Changes in the wall thickness

Fig. 7. Changes in the storey weights

Fig. 8. Distribution of the lateral displacements through the height of the structure
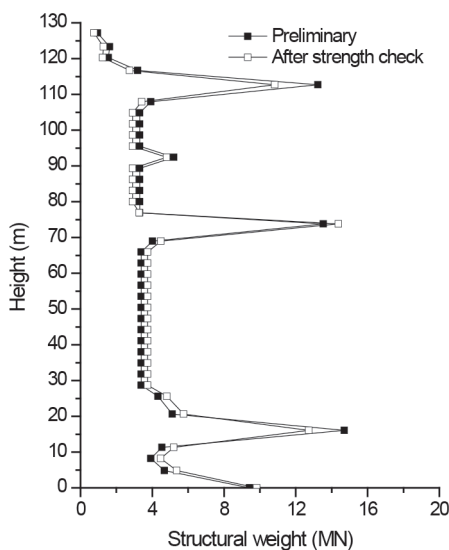
rections are listed in Table 3. The natural frequencies of 0.2252 Hz and 0.2572 Hz are increased to 0.2466 Hz and 0.2792 Hz in the X and Y directions, respectively, showing increases of 9.50% and 8.55%. Therefore, it is expected that the design model with the resizing algorithm will provide a design with controlled wind-induced responses.

3.3. Wind-induced responses

To evaluate the efficiency of the proposed model, the wind-induced responses of the 37-storey building structure before and after the application of the model are estimated by the two methods discussed in Section 1.

For the evaluation of the responses by the simplified equations (NBCC 1995), the density and damping ratio in the X and Y directions of the building are given by 361.98 kg/m^3 and 2%, respectively. The estimated wind-induced vibrations according to the simplified equation in NBCC (1995) are summarized in Table 4. The peak accelerations in both the along-wind and cross-wind directions of 5.92 mg and 7.56 mg are reduced to 5.39 mg and 6.81 mg, respectively.

Table 2. Changes in the lateral displacements at the top in the X and Y directions

	The maximum lateral displacement (mm)		Rate of changes in the maximum lateral displacement (%)	
	X direction	Y direction	X direction	Y direction
Preliminary	215.90	183.70	–	–
After strength check	179.80	159.00	−16.72	−13.45

Table 3. Changes in the natural frequencies of the example by the resizing method

	Natural frequency (Hz)		Rate of changes in natural frequency (%)	
	X direction	Y direction	X direction	Y direction
Preliminary	0.2252	0.2572	–	–
After strength check	0.2466	0.2792	9.50	8.55

After the strength check (step 14), the along-wind peak acceleration is reduced by 8.87 % while the natural frequency is increased by 9.50% in the X direction. The cross-wind peak acceleration is reduced by 9.91% where the natural frequency is increased by 8.55% in the Y direction. These results are in accord with the fact that the level of the wind-induced vibration can be assumed to be inversely proportional to the natural frequency of the building (Tallin, Ellingwood 1984; Griffis 1993).

For the evaluation of the wind-induced vibration by the HFFB method, a wind-tunnel test was performed. A boundary layer wind tunnel with a closed circuit type was used. The width, height and length of the tunnel are 3 m, 2 m, and 20 m, respectively. The range of the wind velocity measureable by the tunnel is 0.5 to 30 m/s. A power law for the profile of the wind velocity was used. The decay coefficient for a decrease in the speed-up with the height is 0.33. The scale of the model is 1/400 and the reproduction range of the surrounding area has a 400 m radius with a design-scale model as the center. The wind-tunnel test was performed 36 times at 10° angle intervals. The reference axis for the angle of the wind direction is shown in Figure 4. The time-varying wind loads were measured by a six-component force balance installed at the bottom of the design-scale model. The sampling frequency was set equal to 200 Hz for a total of 51.2 sec. The maximum hourly mean wind velocity with a return period of 10 years was used for the serviceability evaluation. The roughness and design velocity for the serviceability evaluation were A and 26.76 m/s, respectively. Using the spectrum data obtained from the experiment as shown in Figure 1, the wind-induced response of the building after the strength check (step 14) was evaluated. Figure 7 shows the spectrum data of a wind direction of 150° which generates the highest wind-induced acceleration among 36 directions. The results of the evaluation of the wind-induced vibration for the preliminary design and for the design after the strength check are shown in Figure 9. The peak accelerations of 9.95 mg and 9.69 mg are decreased to 9.60 mg and 8.23 mg, respectively. The peak accelerations are decreased by 3.48% and 15.04% in the X and Y directions, respectively. As shown in Figure 9, the wind-induced vibrations are generally decreased by the proposed optimal model. However, as shown in Figures 9 and 10, the wind-induced vibrations in some directions are increased, indicating that an increase in the natural frequency does not always result in a decrease of the wind-induced vibration. This is due to the property of the wind force spectrum. As shown in Figure 11, although the wind force spectrum of 0.0493 in the X direction is
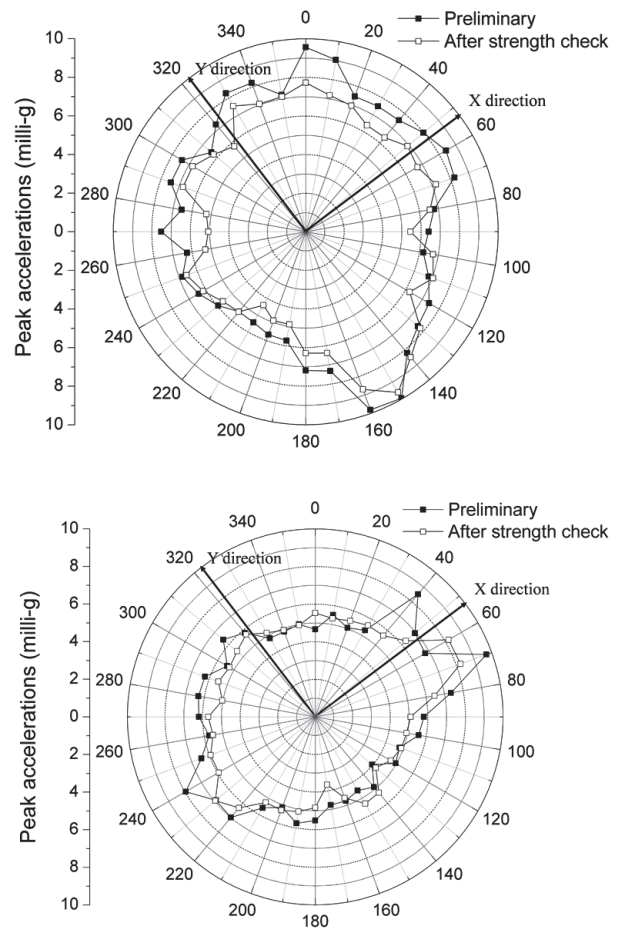

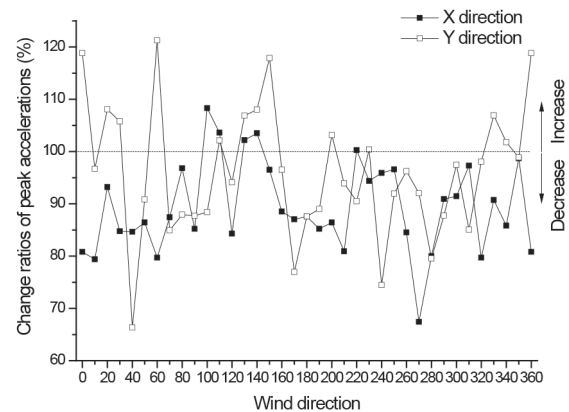Fig. 9. Peak accelerations according to the wind direction

Fig. 10. Reduction ratios of the maximum acceleration after applying the resizing method

Table 4. The results of the wind-induced response obtained from simplified equations in NBCC

	Peak accelerations (mg)		Change of the peak accelerations (%)	
	X direction	Y direction	X direction	Y direction
Preliminary	5.92	7.56	–	–
After strength check	5.39	6.81	−8.87	−9.91

decreased to 0.0425, the wind force spectrum of 0.0135 in the Y direction is increased to 0.0174. Therefore, the peak acceleration of 9.95 mg in the X direction is decreased to 9.60 mg and the peak acceleration of 4.52 mg in the Y direction is increased to 5.33 mg. This is why the wind force spectrum is generally decreased along with the natural frequency, but not always. In contrast, in the case for NBCC 1995, the wind-induced vibration could always be reduced by increasing the natural frequency, as shown in Figure 12.

Conclusions

A practical design model based on the resizing algorithm is presented to control of the wind-induced responses of a high-rise building. In the model, to reduce the wind-induced responses, the stiffness of a high-rise building is maximized to increase the natural frequency of the building by the resizing algorithm. The proposed design model is applied to control the wind-induced vibration of an actual 37-storey building during the initial stage of its structural design.
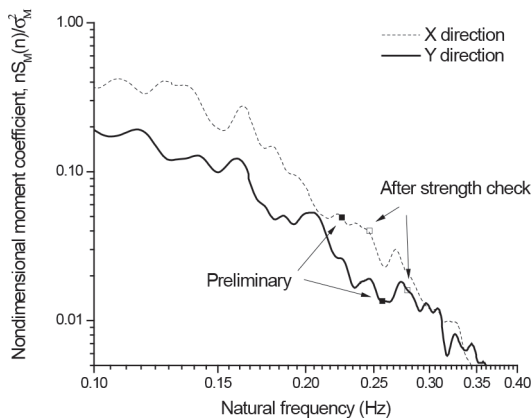

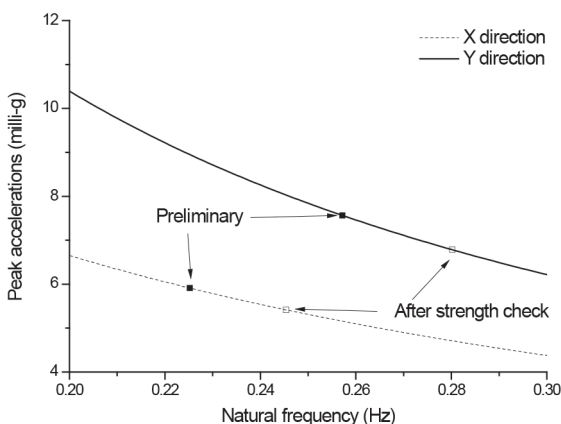Fig. 11. Comparison of the wind force spectrum as measured by HFFB

Fig. 12. Comparison of the wind-induced vibration as evaluated by NBCC 1995

From the evaluation of the wind-induced vibrations by both the simplified method in NBCC and the HFFB method, the wind-induced vibration for an actual 37-storey building was successfully reduced by the design model. Also, contrary to the findings of existing research (Chan, Chui 2006; Chan *et al.* 2009, 2010; Li *et al.* 2011), all of which require an iterative structural analysis, the proposed design model based on resizing algorithm can obtain this effect without an iterative structural analysis.

The wind load applied to the building changes according to the natural frequency of the building. However, this study used the calculation process of the wind load in AIK (2000), which does not consider the influence of the natural frequency of the building. As the wind load applied to a building generally increases along with the natural frequency of the building, considering this effect during the optimization process is expected to result in an additional reduction of the structural weight. A study of the optimal design model for the wind-induced vibration of a high-rise building considering this effect will be carried out as a continuation of research.

Acknowledgments

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