International Journal of High-Rise Buildings
December 2014, Vol 3, No 4, 305-310

International Journal of
High-Rise Buildings
www.ctbuh-korea.org/ijhrb/index.php

Assessment on Natural Frequencies of Structures using Field Measurement and FE Analysis

Do Hyun Kim[1] and Ji Young Kim[2,†]

[1]*Gyeonggi College of Science and Technology, Gyeonggigwagi-daero 269, Siheung 429-792, Korea*
[2]*Daewoo E&C, Sinmunno 1-ga, Hongno-gu, Seoul 110-713, Korea*

Abstract

Wind-induced responses of a structure are often evaluated through dynamic analysis, where measured wind forces obtained from a wind-tunnel test and dynamic properties obtained from a FE (Finite Element) model are utilized. However, the FE model generally shows considerable discrepancies in the estimation of natural frequencies compared to field measurements due to some assumptions and simplifications. In this paper, a calibration method that can improve the estimation of natural frequencies in the FE model is proposed, and specific cases are studied for its validity with comparison to the field measurement results.

Keywords: FE model, Calibration, Dynamic properties, Field measurement, Wind tunnel test

1. Introduction

In order to acquire reliable wind effects to structures, not only wind tunnel test results but dynamic properties of a structure should be accurately evaluated especially for flexible structures. It is because wind-induced responses of such structures are mainly determined from resonant responses which are evaluated with dynamic analysis based upon natural frequencies, mode shapes, and modal damping ratios.

It is generally accepted that damping ratios could be reasonably chosen when structural material of main structural system is determined. Based on previous experiences and measured data, damping ratios are specified in various national codes and guidelines along with equations to estimate natural frequencies, when structural material of main structural system is determined. However, they seem not useful, because lots of structures are designed to be slender, asymmetric, or atypical in these days. Rather natural frequencies and mode shapes are often evaluated through FE analysis models.

Current field measurements have revealed that analyzed natural frequencies could be quite different compared with actual ones (Kim et al., 2009). This is mainly because some assumptions and simplifications are included in FE models which are constructed based on structural information at design develop stage. Such improperly derived natural frequencies could inevitably make critical errors in evaluating wind-induced responses especially for flexible structures. The discrepancies in natural frequencies imply that stiffness of a structure is improperly evaluated. This means that static wind-induced responses (mean and background responses) could be distorted.

It is important to evaluate reliable prediction of wind-induced responses especially in design of tall and large span structures. It is mainly because structural system is generally determined to prevent serviceability problems due to wind loads. Vibration control devices could be additionally used to reduce wind-induced responses. Construction costs could be considerably increased due to over-estimated wind-induced responses which are derived using under-estimated natural frequencies. Such under-estimations of natural frequencies are usually caused by neglecting contributions of non-structural members, increase of elastic modulus of in-situ concrete to overall stiffness of structures in FE models.

To minimize such improper structural designs, considerable efforts have been made in field measurements for tall and long span structures by Daewoo E&C since 2006 (Kim, 2007). The FE models were calibrated based on the measured dynamic properties for serviceability evaluation. In addition, measured wind-induced responses of a building and a roof were compared with the wind tunnel test results to investigate importance of accurate dynamic properties.

2. Field Measurement and System Identifications

On-site vibration measurements were performed for a series of buildings and a large span roof. The types of structures are summarized in Table 1. Structures B1 to B6

†Corresponding author: Ji Young Kim
Tel: +82-31-250-1163; Fax: +82-31-250-1132
E-mail: jiyoung.kim@daewooenc.com

Table 1. Types of structures

Name	B1	B2	B3	B4	B5	B6	R7
Type of structures	Building						Roof
Structural system	RC core & moment frames		RC core, moment frames & Outrigger walls		RC core & flat plate	Shear walls	Steel truss
Height (m)	142	113	134	140	119	57	*Span 200m

in Table 1 are residential buildings which show a typical type of architectural and structural plans in Korea. The laterally-resisting structural system of the building B1 and B2 is composed of RC cores and moment frame. The center core is connected with columns using diaphragms and columns are connected between each other using spandrel beams in typical floors of the B1. Core and external columns are connected each other with beams in typical floors of the B2. Structures B3 and B4 is composed of RC cores and moment frames similar to the building B1 and B2. They, however, have outrigger walls at the top floors of the buildings to increase lateral stiffness.

The laterally-resisting structural system of the building B5 is composed of RC cores, columns, and flat-plates. This means that slabs of the building B5 are designed to contribute lateral stiffness of it. The B5 is composed of 2 towers which are connected with each other using a sky bridge at 10^{th} floor. The sky bridge is designed to be disconnected at the center of its span using expansion joints. At design develop stage, the FE model of the B5 is developed as separated towers to consider discontinuity between them due to the expansion joint of the sky bridge.

Although the building B6 is categorized into the building structure, it shows somewhat different structural system compared with residential tall buildings from B1 to B5. The B6 is a typical type of apartment which is popularly constructed in Korea. The laterally-resisting system of it is mainly composed of shear walls. Also, design of lateral system is usually governed by seismic loads because the height is generally low. The R7 is a long span roof structure composed of steel trusses suspended by mast cables from main masts.

The accelerations of the chosen structures were measured to derive dynamic properties such as natural frequencies, mode shapes, and damping ratios. The wind velocities corresponding to wind directions were also measured to investigate response amplitudes corresponding to wind velocities. Total 24 accelerometers were installed on 8 floors to measure wind-induced vibrations. Three accelerometers were located on each floor to identify two translational components and a rotational component of building motion. The accelerometers were moved to the next measurement points after one measurement setup is finished except for three accelerometers at top floor which are designated as reference points (Fig. 1). Two data loggers were used for data acquisition and the GPS time stamps were used to synchronize the measured data.

For building B5, accelerations were measured only at the top of the 2 towers to identify coupled motion of them. Total 8 accelerometers were installed on the top floors and 4 accelerometers were located on each tower to measure translational and rotational motions. Fortunately, typhoon-excited vibrations could be measured for the building B3. For the roof R7, long-term monitoring system was installed since 2006 and wind-induced responses due to typhoons, strong wind, etc. could be measured.

The dynamic properties of each structure were evaluated using the Frequency Domain Decomposition (FDD)

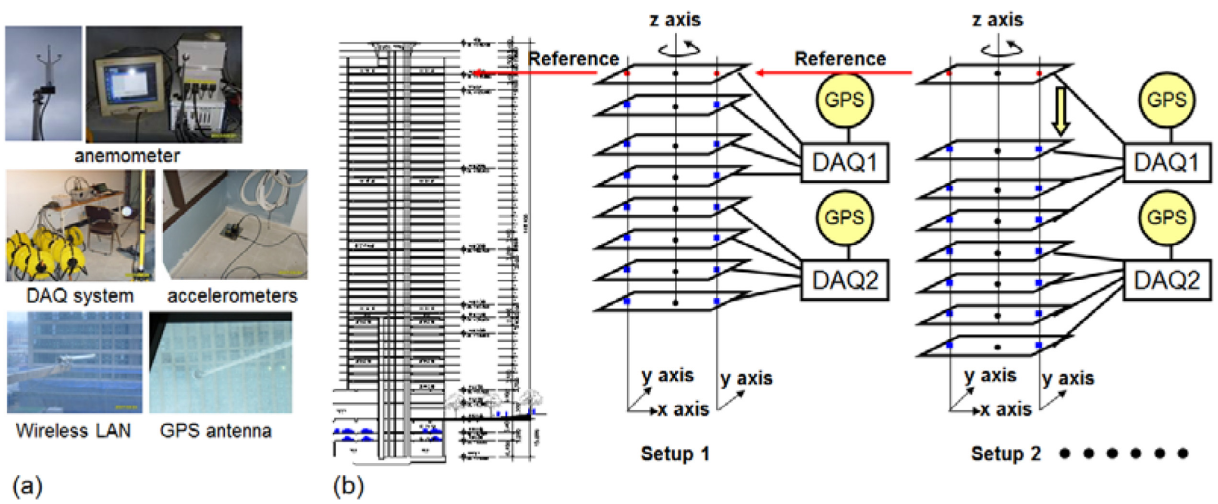


Figure 1. Measurement system: (a) equipments, (b) measurement method.

Table 2. Natural frequencies

Classification	Number	1	2	3	4	5*	6	7
Measurement	1st mode	0.356	0.328	0.356	0.256	0.352	1.190	0.745
	2nd mode	0.381	0.360	0.415	0.349	0.455	1.203	0.874
	3rd mode	0.713	0.609	0.811	0.625	0.687	1.920	0.950
Original FE model	1st mode	0.226	0.232	0.236	0.252	0.302	0.627	-
		(63%)	(71%)	(66%)	(98%)	(86%)	(53%)	-
	2nd mode	0.282	0.238	0.315	0.260	0.440	0.875	-
		(74%)	(66%)	(76%)	(74%)	(97%)	(73%)	-
	3rd mode	0.461	0.337	0.399	0.455	0.575	0.961	-
		(65%)	(55%)	(49%)	(73%)	(84%)	(50%)	-
Calibrated FE model	1st mode	0.350	0.341	0.336	0.255	0.339	1.192	0.717
		(98%)	(104%)	(94%)	(100%)	(96%)	(100%)	(96%)
	2nd mode	0.372	0.356	0.434	0.338	0.497	1.252	0.847
		(98%)	(99%)	(105%)	(97%)	(109%)	(104%)	(97%)
	3rd mode	0.591	0.551	0.734	0.516	0.669	1.796	0.934
		(83%)	(90%)	(91%)	(83%)	(97%)	(94%)	(98%)

*The 1st, 3rd, and 5th modes

method (Brincker et al., 2000). Using the FDD method, the natural frequencies can be picked from the Singular Value (SV) plot. Also, the mode shapes can be extracted from the Singular Vectors corresponding to the natural frequencies. Damping ratio of each vibration mode is estimated from the Auto Spectral Density Function by transforming it to a free decaying signal. The measured natural frequencies for each structure are summarized in Table 2.

3. Comparisons of Natural Frequencies

3.1. Building B1 to 4

It could be found in Table 2 that the natural frequencies evaluated with the original FE models are quite lower than the measured ones. The Modal Assurance Criteria (MAC), however, shows a good correlation between measured and analyzed mode shapes. This implies that the discrepancies are not originated from local modeling errors in FE model but from global modeling mismatches (Fig. 2). It, therefore, is presumed that the discrepancies are mainly caused by the followings (Kim et al., 2009a):

a. Elastic modulus of in-situ concrete
b. Non-structural members: cement brick walls, plain concrete walls, parapets, etc.
c. Flexural rigidity of slabs
d. Beam-end-offset

The elastic modulus of in-situ concrete is about 10% larger than the specified values because compressive strength of it is generally increased due to concrete quality control in manufacturing process. The non-structural components considerably contribute to the overall stiffness in serviceability vibration level. In FE models of moment-frame structures, the slabs are occasionally neglected based on diaphragm assumption. They, however, show some contribution to the lateral stiffness. Also, the effect of beam-end-offset could be noticeable depending on the geometric configuration of beams and columns.

When above factors are considered in the FE models, the discrepancies between measured and analyzed natural

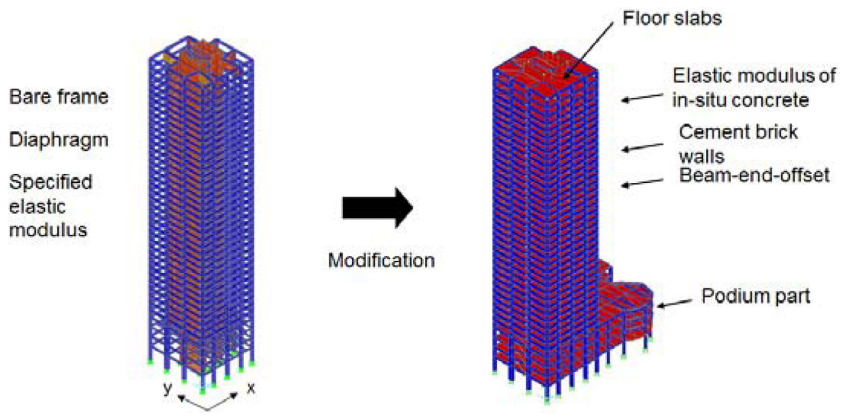


Figure 2. Calibrations of FE models.

frequencies were observed to be considerably decreased as shown in Table 2. By considering beam-end-offset, natural frequencies were observed to be increased by 1~6%. Natural frequencies were increased by 3~11% by including flexural rigidity of slabs in FE models. The floor slabs of the B1 to B3 were modeled using plate elements. In case of the building B4, however, the effective beams were used instead of plate elements to consider flexural rigidity of slabs in FE models. By increasing elastic modulus of concrete material in FE models, natural frequencies were observed to be increased by 7~12%.

The effect of non-structural walls to natural frequencies was observed to be influenced by the geometric configurations in planar plan. For the building B1 and B3, natural frequencies were increased by 5%-12% by considering non-structural walls. There was not a non-structural wall in the building B2. In case of the building B4, the natural frequency of the 2nd mode (x-axis) shows 26% discrepancies with the measured results while that of the 1st mode show 2% discrepancies. In the building B4, site investigation reveals that cement brick walls were installed along the x-axis. After considering the cement brick walls, the discrepancy of natural frequency of the 2nd mode is decreased to 3%.

3.2. Building B6

For the building B6, the FE model was calibrated by considering elastic modulus of in-situ concrete and flexural rigidity of floor slabs. The beam-end-offset was not considered because structural system is composed of only shear walls and floor slabs. After that, parapets on the balconies were additionally modeled in the FE model of the B6. Basically, balconies are classified into non-structural components in design practices. They, therefore, are normally neglected in FE models in design develop stage.

Balconies are generally installed between shear walls for the shear-wall type apartment buildings in Korea. Inserted balconies between shear walls could make considerable contribution to lateral stiffness similar to embedded brick walls in frames. In this FE model calibration, balconies were modeled using equivalent beams between shear walls. After considering balconies, the discrepancies of natural frequencies of the B6 were observed to decrease from 47% to 6%.

3.3. Building B5

For the building B5, beam-end-offset, non-structural walls, and elastic modulus of in-situ concrete were additionally considered in FE model construction. Floor slabs were already modeled in design develop stage because the B5 was designed to resist lateral loads with columns connected with cores using thick plates.

As shown in Fig. 4(a), two towers of the B5 are connected with each other by a sky bridge at the 10th floor. At design develop stage, the towers were assumed to be separated and a FE model for only an independent tower was constructed. Preliminary measurements for the B5, however, showed that fundamental modes could not be identified using measured data only for an independent tower. The accelerations, therefore, were concurrently measured on the top floors of the 2 towers to measure coupled motions of them.

Basically, the sky bridge was planned to be separated using expansion joints. The expansion joints, however, are filled with adhesion of filler material. The filler material, however, has considerable strength and stiffness. It is presumed that the filler material could firmly connect separated two parts of the sky bridge within serviceability vibration level. Fig. 3(b) indicates that the stiffness of filler material is enough to induce coupled mode shapes between two towers. The FE model of the B5, therefore, was

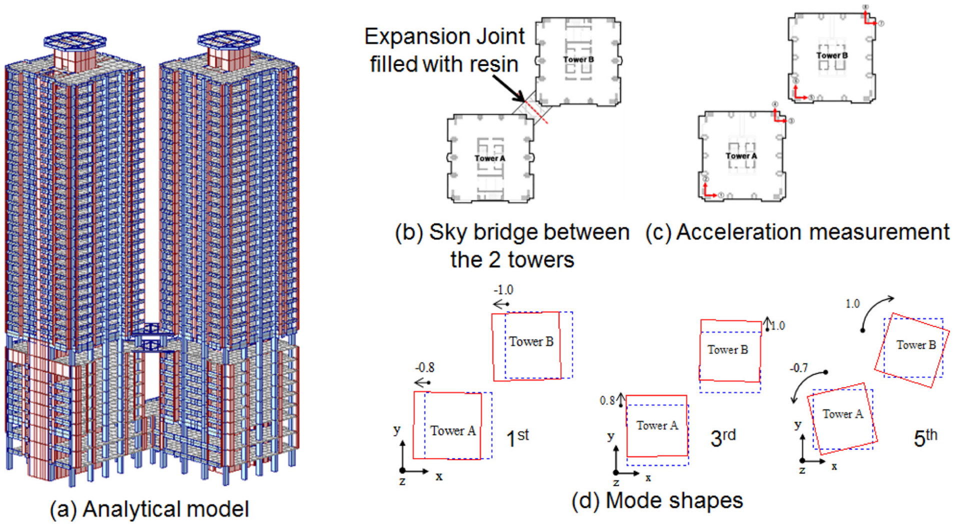


Figure 3. Coupled behaviors of Building No. 5.

modified so that two towers are connected using link elements between the joints of the sky bridge. After calibration, the natural frequencies of the calibrated FE model become closer to the measured ones.

3.4. Roof R7

Wind-induced vibrations were measured for the roof R7 since 2006 using permanent Structural Health Monitoring (SHM) system. The natural frequencies of it were evaluated using the measured acceleration data and were compared with the analyzed results of the FE model in Table 2. As shown in Fig. 4, the structure of the roof is composed of main steel trusses and membranes. In case of the R7, Membrane is a secondary structural element which could be regarded as cladding. Hence, only the steel trusses, cables, and main masts are considered in FE model. The mass of mechanical and acoustic devices were investigated and added in the FE model as nodal masses.

The roof trusses were fabricated on temporary supporting structures during construction. After completion of the roof trusses, they were suspended from main masts using stay cables. Before removing temporary steel structure, prestress was introduced in the main mast cables. To accurately evaluate natural frequencies of the R7, such construction sequence was considered in FE model analysis. Firstly, prestress was introduced in link elements shown in Fig. 4(b). After that, a static analysis was performed for applying self weight of the steel trusses. Finally, modal analysis was performed to derive natural frequencies and mode shapes.

It was observed that natural frequencies were not significantly changed according to whether construction sequence is considered or not. It seems because overall stiffness is not considerably changed although geometry of steel trusses in FE model constructed prior to modal analysis step could be somewhat changed after the static analysis corresponding to construction sequence. Unlike the building structures, it could be also observed in Table 2 that the natural frequencies of the roof could be accurately evaluated only with the minor modification of mass distribution.

4. Damping Ratios

Fortunately, large amplitude of dynamic responses could be measured for the building B3 and the roof R7 when typhoons stroke them. Using the FDD method, modal damping for the building and roof was evaluated. Damping ratios are observed to show amplitude dependency as damping predictors proposed by AIJ (2009), Jeary (1986), and Yoon (2002). The maximum damping for the tower is close to 1.5% at maximum measured response.

Damping ratios for long span roofs have not been proposed as a damping predictor because there seems not enough measured data for various roof structures. The maximum measured damping ratio is about 1.5%. It is somewhat larger than currently accepted values 0.5~1% for steel structures. This seems because aerodynamic damping is included in total damping.

5. Conclusions

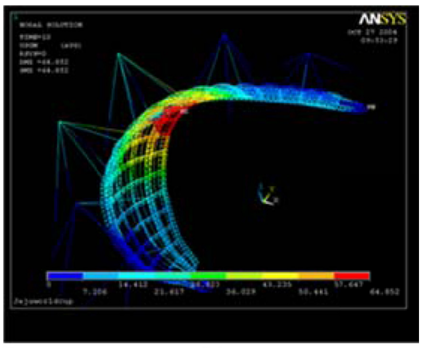
From a series of comparisons for dynamic properties and wind-induced responses, it could be confirmed that current FE models of structures in practical engineering are required to be calibrated to acquire more accurate wind tunnel test results. The proposed factors for the calibrations of FE models for building type structures could be quite effectively used in development of FE models in practical engineering and research areas. The proposed calibration approaches, however, could be effective only for specific buildings and roof structures. More field measurements and calibration studies, therefore, are required in future studies to generalize procedures of FE model constructions.

References

Architectural Institute of Japan (2000) Damping in Buildings. Architectural Institute of Japan.

Jeary, A. P. (1986) "Damping in Tall Buildings - a Mechanism and a Predictor." *Earthquake Engineering and Structural Dynamics*, 14, pp. 733~750.

(a)



(b)

Figure 4. Structural system of roof 7.

Kim, J. Y., Yu, E. J., Kim, D. Y., and Kim, S. D. (2009a) "Calibration of Analytical Models to Assess Wind-induced Acceleration Responses of Tall Buildings in Serviceability Level", *Engineering Structures*, 31, pp. 2086~2096.

Kim, J. Y., Cho, J. O., Kim, D. Y., Yoshida, A., Tamura, Y., and Kim, S. D. (2009b) Investigation of Dynamic Properties for a Large Span Roof by Comparing Results of Field Measurement and FE Analysis, *Proc. the 4th International Symposium on Steel Structures (ISSS)*, Seoul, Korea, pp. 428~432.

Brincker, R., Zhang, L., and Andersen P. (2000) "Modal Identification from Ambient Responses using Frequency Domain Decomposition." *Proc. the 18th International Modal Analysis Conference (IMAC)*, San Antonio, Texas, USA, pp. 625~630.

Yoon S. W. (2002) "Damping Ratios of Reinforced Concrete Apartment for Serviceability Design of Wind," *Journal of Architectural Institute of Korea*, 20, pp. 27~34.

Xie J. and Irwin P. A. (1998) "Application of the Force Balance Technique to a Building Complex," *Journal of Wind Engineering and Industrial Aerodynamics*, 77&78, pp. 579~590.