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The Construction of Initial Analytical Models Structural Health Monitoring of a Masonry Structure

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Abstract

It is important to accurately predict structural responses to external excitations such as typhoons and earthquakes when designing structures for serviceability. One of the key procedures to predict reliable vibration responses is to evaluate accurate structural dynamic properties using finite element (FE) models, which properly represent the realistic behavior of buildings. In the case of historic masonry buildings, structural damage could also be caused by ambient vibrations or impacts. Therefore, the preservation plans of historic buildings for low-level vibrations or impacts should be provided by analyzing structural damages within serviceability levels. For this purpose, it is required to provide FE model construction and response analysis methods verified with field measurement data. In this research, long-term field measurement was performed for a cathedral and its dynamic properties were evaluated using measured data. Then, the model was calibrated based on the measured dynamic properties and an overall construction method for the masonry cathedral was proposed. Using the measured accelerations, the vibrations of the belfry were analyzed using the calibrated FE model and finally, the FE model for the cathedral was verified by comparing the measured accelerations with the modeled results.

Keywords: Cathedral, Vibrations, Field measurement, Wind, Finite element model, Calibration

1. Introduction

One of the important processes to accurately predict structural responses to external excitations such as typhoon, earthquake, etc. is to evaluate structural dynamic properties accurately using finite element (FE) models which properly represent buildings' realistic behaviors. The analyzed structural dynamic properties of tall buildings usually show considerable discrepancies with the measured ones because structural details are simplified in the FE models to perform practical design of structures. For example, non-structural elements such as light-weight concrete partition walls, parapets, etc. are neglected in the FE models. Floor slabs are usually assumed as diaphragms which neglect flexural stiffness of them.

To compensate such discrepancies, FE models for tall buildings have been calibrated based on the reference dynamic properties which were evaluated using field measurement data. Modeling conditions of FE models for tall buildings are changed until the analyzed dynamic properties show a good agreement with the measured ones. Then, a series of calibration process of FE models have been proposed for tall buildings in previous researches (Fu et al., 2008; Kim and Kim, 2014; Kim et al., 2009).

Also, long-term monitoring for a long-span roof were performed, the variations of structural dynamic properties due to external environmental effects were investigated using measured acceleration data (Kim et al., 2011). The structural analysis model was also calibrated based on the measured dynamic properties. In addition, it was demonstrated that wind-induced responses could be reliably predicted using an accurate FE analysis model and proper wind tunnel tests.

Analytical and experimental researches have been performed for masonry structures to evaluate large deformations and structural damages during strong excitations such as earthquakes (Chengqing et al., 2005; Park et al., 2009). The structural performance and fragility of masonry structures subject to earthquakes have been investigated based on the developed analytical methods (Sima, J. F., 2011). In these previous researches, nonlinear FE models, equivalent components or FE elements for masonry structures have been mainly developed to analyze earthquake behaviors of masonry buildings.

Masonry buildings, like tall buildings, also frequently experience ambient vibrations within serviceability level due to winds, vehicles, heavy construction equipment, etc. These ambient vibrations could lead to gradual damages in masonry buildings. It, therefore, is necessary to provide maintenance guides or criteria within serviceability level

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for the masonry structures by analyzing their structural behaviors including the ambient vibrations. Regarding earthquakes, structural performances under the immediate occupancy level need to be analytically evaluated in performance-based seismic design for masonry buildings [FEMA, 1998].

In previous researches, however, analysis of structural responses and their verifications through field measurements for the masonry buildings have not been intensively performed. Moreover, few field measurement data within the serviceability level are available for the masonry buildings. Therefore, calibrated FE models within serviceability level are not easily found in previous researches.

Historic masonry buildings such as royal palaces, cathedrals, etc. are important cultural heritages and should be preserved in the future. As an effort of preservations, it is required to understand their structural behaviors. Also, special structural forms such as domes, arches, belfries, etc. of cathedrals especially attract researchers' interests to investigate them. Several researches, therefore, have been performed to investigate structural behaviors of the historic buildings using analytical methods (Cagnan, 2012; Lourenço et al., 2012; Mallardo et al., 2008). Since earthquakes are generally regarded as the most possible cause of damages in such historic masonry structures, analysis methods of large deformations due to strong earthquakes have been developed in most researches to investigate seismic damage and fragility of the structures.

In case of the historical masonry buildings, structural damages could be also caused by ambient vibrations or impacts. Preservation plans, therefore, of the historic buildings for low level vibrations or impacts should be provided by analyzing structural damages within serviceability level. For this purpose, it is required to provide FE model construction and response analysis methods verified with field measurement data.

In this research, long-term field measurement is performed for a cathedral building and its dynamic properties are evaluated using measured data. A basic FE model is constructed for the cathedral using the ABAQUS/Explicit (HKS, 2012), structural analysis tool. Then, it is calibrated based on the measured dynamic properties and overall construction method of FE models for masonry cathedral is proposed. Also, it is investigated whether the current evaluation method of wind-induced responses can be practically applicable for masonry buildings by comparing measured accelerations at the belfry with analyzed results. Also, belfry experienced vibrations induced by the construction works at an adjacent construction site were also measured during the long-term monitoring. Using the measured accelerations at the base, the vibrations at the top of the belfry were analyzed using the calibrated FE model for it. Finally, the FE model for the cathedral could be verified by comparing the measured accelerations with the analyzed results of the FE model.

2. Target structure

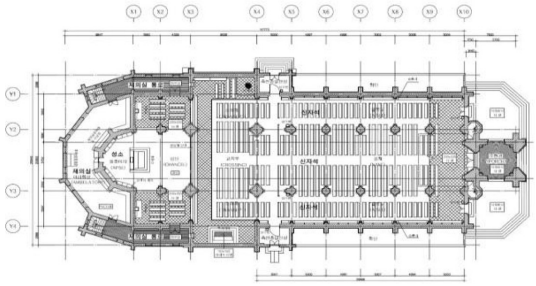
The Target structure is a Roman Catholic cathedral which is a historic building built in unreinforced masonry located in Seoul, Korea. Fig. 1(a) to (c) show the perspective view, an architectural plan, and an elevation, respectively. Currently, it is the religious facility which is established 117 year ago and is registered as a historical landmark No. 258 of Korea. The height of a belfry is 50.45 m.

3. Installation of Vibration Measurement System for Masonry Structure

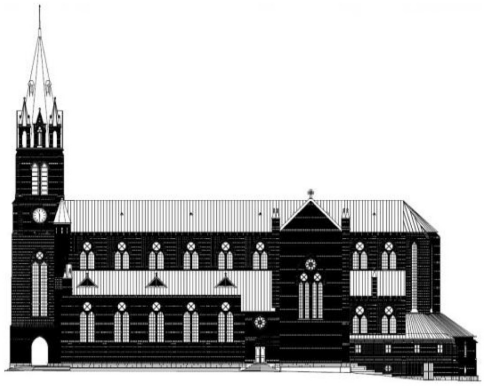
Acceleration responses of the belfry and ground excitations were measured using servo type accelerometers as



(a) Perspective view



(b) Architectural plan



(c) Elevation

Figure 1. Perspective view, architectural plan, and elevation of the target building.

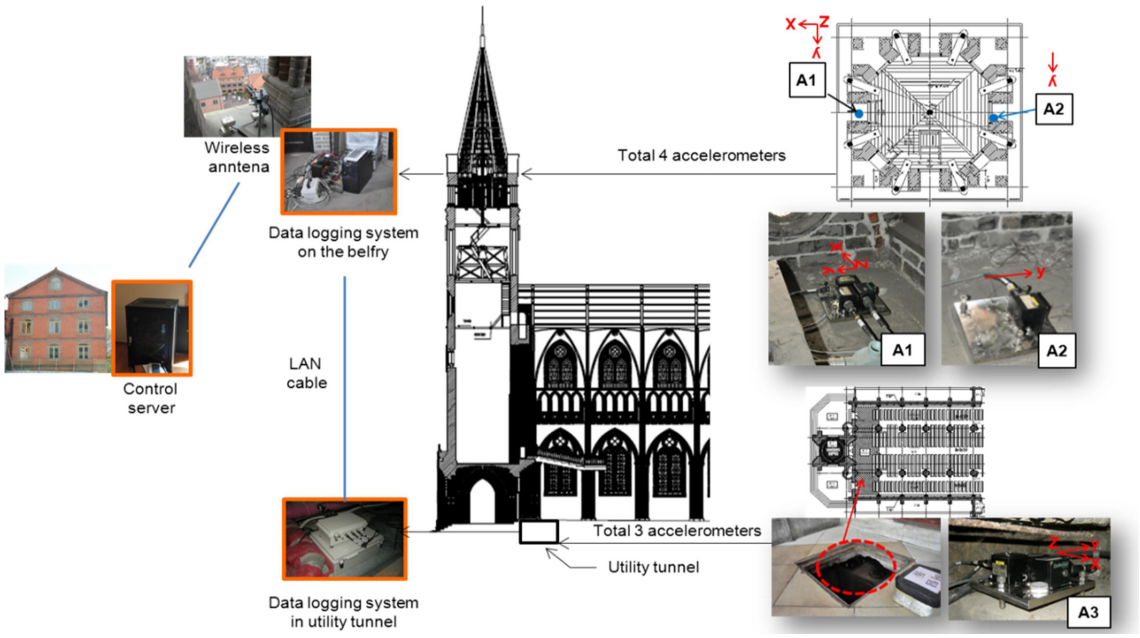


Figure 2. Installations of accelerometers.

shown in Fig. 2. Vibrations from a construction site and a parking lot near the cathedral were expected to be transferred to it during the measurement. Accelerometers were installed on the ground level of the cathedral to measure the excitation vibrations. Since the acceleration responses were expected to be the most significantly amplified at the top of the belfry, accelerometers were placed on it. Total of three accelerometers were installed on the top floor of the belfry. Two of them were installed at the location A1, to measure the response accelerations along the x- and y-axis as shown in Fig. 2. The other one was placed at point A2, separated from A1 to measure rotational responses about z-axis. Also, three accelerometers were installed at point A3, in the utility tunnel under the belfry to measure the ground accelerations along the x-, y-, and z-axis.

Acceleration data were measured at the rate of 200 Hz and were filtered using analogue low-pass filters to remove aliasing noise in signals.

4. Evaluation of Dynamic Properties

The dynamic properties of the belfry such as natural frequencies, mode shapes, and damping ratios were extracted from the acceleration data measured at the top floor of it.

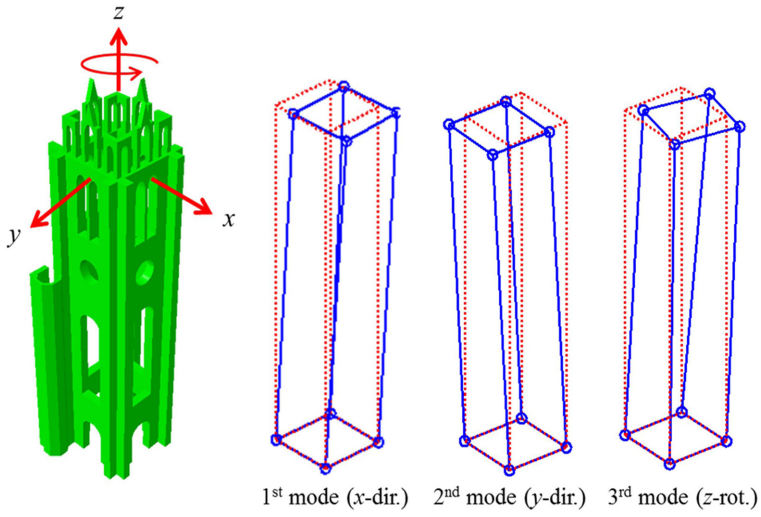


Figure 3. Measured mode shapes.

Table 1. Measured natural frequencies and corresponding damping ratios

Mode	1st	2nd	3rd
Axis	*x*-dir.	*y*-dir.	*z*-rot.
Natural frequency[(a)]	1.48	1.64	1.82
Damping ratio[(b)]	0.59%	0.90%	0.80%

Noise signals such as impacts, etc. are intermittently observed in the middle of response accelerations due to constructions and traffics near the belfry. Since the results of the system identification (S.I.) could be distorted due to the contaminated accelerations, The S.I. was performed using samples of accelerations which were measured when the external excitations were little. In addition, the measured data were filtered using the low-pass filter at the cut-off frequency of 20 Hz to remove high frequency noises which has little correlation with structural responses.

The frequency domain decomposition (FDD) method was used for the S.I. process in this research (Brincker et al., 2001; Brincker et al., 2000). The FDD is one of modal identification methods of building structures with unknown excitation forces and can be effectively used for buildings subjected to wind force and ambient vibration. Total of three dynamic modes are picked from the SV plot and their natural frequencies are 1.48 Hz, 1.64 Hz and 1.82 Hz, respectively. The mode shapes are calculated from the singular vectors. It is observed in Fig. 3 that the 1st and 2nd modes are translational mode shapes along the y- and x- axis, respectively. The 3rd mode is observed as a rotational mode about z-axis in Fig. 3.

The damping ratios are summarized in Table 1 and are 0.59%, 0.90% and 0.80% for the lowest 3 modes, respectively. The measured damping ratios are much lower than generally known values for low-rise buildings. It is generally accepted that damping decreases as response decreases due to the amplitude dependency of damping. The reason why damping ratios are much low in the belfry is that they are evaluated in ambient vibration level (i.e., low amplitude vibration).

5. Finite Element Modeling

As mentioned in the introduction, the material and geometric finite element analyses were performed using ABAQUS/Explicit (HKS, 2012) to examine the validation of the finite element modeling method in order to investigate the serviceability of the unreinforced masonry building of the ancient Cathedral adjacent to the top-down construction site, and to analyze the structural dynamic characteristics. Fig. 4 displays the finite element modeling and mesh of the target building in this study. The unreinforced masonry building consists of clay bricks and mortar, and is modeled as a unified building using eight-node hexahedral solid elements. The boundary conditions of the building are assumed to be fixed to the ground.

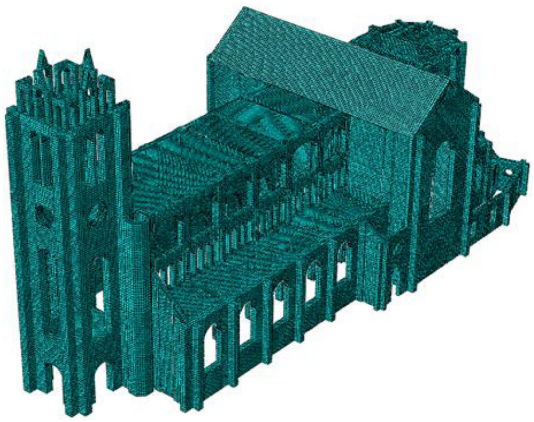


Figure 4. Finite element model and mesh.

KBC (Korean Building Code) 2009 (AIK, 2009) provides two methods to check the compressive strength of masonry, which are a prism test made up using a number of masonry and a strength test of the brick masonry. They can be applied during pre-construction and construction. The prism test can be adopted even when the compressive strength of masonry in the existing building is determined. For reference, KBC 2009 recommended that the compressive strength of masonry of the existing building should be checked through the prism test to get the three masonry prism for each wall area of the building of 500 m^2 after the masonry is 28 days old or older.

Based on KBC 2009, as the target building of this paper is the existing building, the compressive strength of the masonry of the building should be checked through the prism test. However, since the prism test adopts the 28-day compressive strength of the masonry, it is not reasonable to reflect this criterion to the building that was completed 115 years ago from now (September, 2013). Moreover, as the target building is a cultural asset of No.258 of historic sites of the Korea, it is difficult to take specimen for the prism test, since it is possible to cause structural damage of the resisting masonry wall on taking specimen from the existing masonry wall. In this paper, the compressive strength of the masonry is obtained from the compressive strength test of the brick masonry after taking the specimen in an arbitrary place of the target building.

However, despite the confirmation work of the compressive strength through the prism test and the strength test of the brick masonry, information for estimating the compressive strength of the brick masonry of the target building is unavailable. Therefore, in this paper, the compressive strength used in the finite element model was adopted from the value that best matches the dynamic characteristics obtained from the values such as the prism test results, the test results of the brick masonry, and the expected properties provided by FEMA 306 (1998) which is the guideline to introduce the reasonable seismic performance evaluation method of the masonry wall.

Table 2. Test results obtain from the compressive strength of the brick masonry

Specimen	Compressive strength (MPa)	Remark
Brick masonry of No.1~3	10.3	Average value
Masonry prism	5.73	-

Table 4. Comparisons of natural frequencies

Mode	1st	2nd	3rd
Axis	*x*-dir.	*y*-dir.	*z*-rot.
Measured natural frequencies[(a)]	1.48	1.64	1.82
Numerical natural frequencies[(b)]	1.42	1.58	2.28
(a)/(b) × 100 (%)	104.23	103.80	79.82

Table 3. Expected properties of the clay masonry per FEMA 306 (1998)

Condition	Expected masonry compressive strength (f_{me}, MPa)	Expected elastic modulus ($E_{me} = 550f_{me}$, MPa)	Expected shear modulus ($G_{me} = E_{me}$, MPa)	Poisson's ratio
Good	6.21	3,415.5	1,366.2	
Fair	4.14	2,277.0	910.8	0.25
Poor	2.07	1,138.5	455.4	

The strength test results of the prism and brick masonry specimen of the target building and the expected compressive strength of the brick masonry per FEMA 306 closest to the field measurement results are summarized in Table 2 and 3, respectively. The compressive strength, which is obtained from the prism test and the compressive strength test of the brick masonry, is 10.3 MPa and 5.73 MPa, respectively. For reference, the expected properties of the brick masonry provided by FEMA 306 are classified according to the condition of the masonry wall (the good, fair, and poor condition). The expected compressive strengths of the masonry wall with the good, fair, and poor condition are 6.21 MPa, 4.14 MPa, and 2.07 MPa, respectively.

The target building of this paper is a building more than 100 years old and several renovation have been made from after completion. By synthesizing these terms, the state of its masonry wall is determined as "Fair" according to the state of the masonry wall presented in FEMA 306. As described above, the compressive strength of the brick masonry of the finite element model closest to the field measurement comparing the dynamic measurement results and the numerical results was applied in this study. The comparison showed that the compressive strength of the masonry of 4.14 MPa (Fair condition) per FEMA 306 was very similar to the filed measurement result. This is a conservative value compared with the strength test of the brick masonry and the prism test results. Therefore, this value was defined as the compressive strength of the masonry of the building analyzed in this study.

Based on these data, eigenvalue analysis results and mode shapes obtained from the numerical analysis for the belfry of the target building were derived. The 1st, 2nd, and 3rd natural frequencies according to the eigenvalue analysis of the target building were 1.42 Hz, 1.58 Hz, and 2.28 Hz, respectively. The comparison results of the dynamic measurement results and the eigenvalue analysis results of the numerical analysis for the belfry of the target building, mode shapes presented that the field measurement results and the numerical results show very similar tendency. The natural frequency of the 1st and 2nd modes showed considerably similar tendency within the error range of 5% compared to the field measurement results, but the 3rd natural frequency displayed error of about 21% (see Table 4).

6. Wind-Induced Responses

The typhoon Bolaven has influenced Korea in August 2012. It went up to the north along the West coast of Korea and the peak 10-minute mean wind speed was recorded as 13.7 m/s at Seoul meteorological station on 28th August at 11:43 a.m. The wind velocities and corresponding wind directions were also measured at Jung-gu auto weather station (AWS) near the cathedral.

The measured wind data are shown in Fig. 5. The peak 10-minute mean wind velocity and corresponding wind direction measured at the AWS are 9.2 m/s and 53.3, respectively, during the influence of Bolaven.

In Fig. 5(b), the mean wind direction was appeared as approximately 53 while the typhoon was approaching Seoul region. It, therefore, is presumed that wind blew from the northeast at the site of the cathedral and was approximately aligned with the y-axis of the belfry. During the approaching period, y- and x-axis are regarded as the along- and across-wind directions of the belfry, respectively.

During the typhoon, accelerations were measured at A1 location in Fig. 2 and their peak accelerations for 10-minute evaluation time are plotted in Fig. 6. It appears in Fig. 6 that accelerations started increasing after 27th August in the evening as wind velocities increased. Then, the accelerations reached to their peaks on the next day, 28th August at noon and decreased after the typhoon passed. The maximum peak accelerations were recorded as 27.01 cm/s^2 and 18.05 cm/s^2 for x- and y-axis, respectively.

It is investigated whether or not wind-induced responses for a historic masonry structure could be reasonably and reliably evaluated by comparing measured and analyzed results. Since a wind tunnel test was not performed for the cathedral, wind-induced accelerations were evaluated

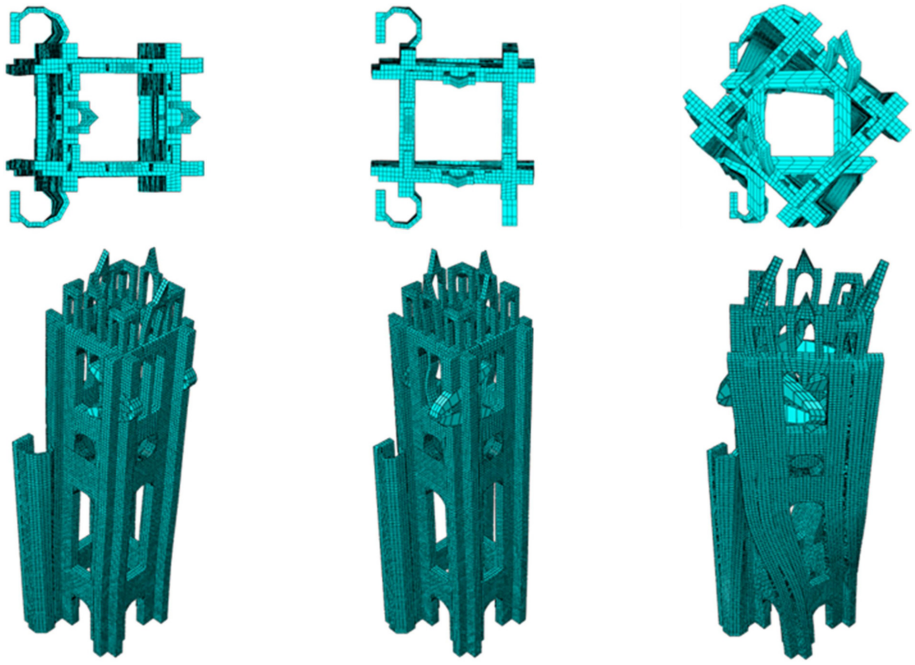
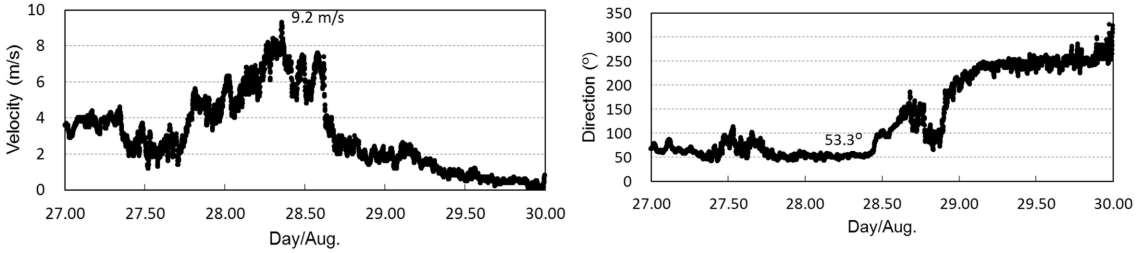


Figure 5. Mode shapes and natural frequencies obtained from the eigenvalue analysis of the finite element model.



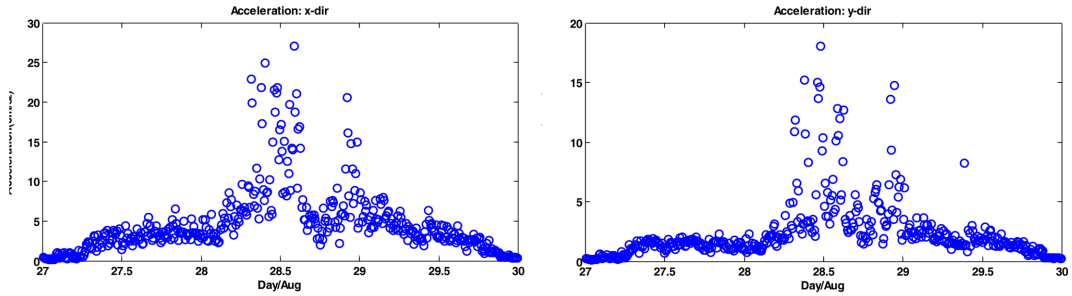
(a) wind velocities (b) wind directions

Figure 6. 10-minute mean wind velocities and corresponding wind directions at Seoul Jung-gu AWS during the influence of Typhoon Bolaven (reference height= 10 m).

in accordance with a procedure provided in NBCC provision (NBCC, 2005). Wind-induced accelerations of a building are evaluated through the stochastic spectral analysis which requires force spectra and dynamic properties. As input forces for the spectral analysis, the wind force spectra for a series of prismatic buildings given in the NBCC procedure are used. The measured natural frequencies and modal damping ratios are used in the evaluations of wind-induced vibrations. The structural mass for the belfry is assessed from the calibrated FE model.

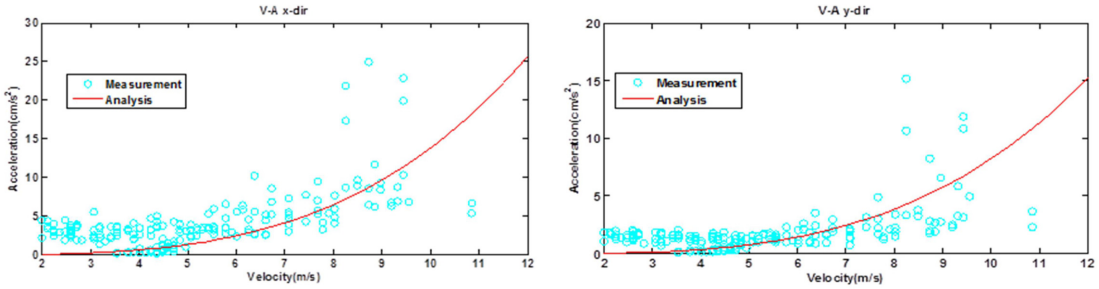
Since wind brew approximately along the y-axis of the belfry, the wind-induced vibrations for the along and the across wind directions are evaluated for the y- and x-axis, respectively. The analyzed accelerations are compared with measured ones as shown in Fig. 7. The measured accelerations corresponding to wind velocities are plotted with circle markers in the graphs of Fig. 8. Wind velocities measured at Jung-gu AWS are used to plot the acceleration-wind velocity graphs. Also, the analyzed accelerations corresponding to the wind velocities are plotted with a line in each graph.

It can be observed in Fig. 8 that the analyzed wind-induced accelerations show a tendency to generally follow the measured. The measured accelerations, however, show some deviations from the analytical results and appear to have some variances in high wind velocity region. One of the main reasons is that the structural constraints and surrounding conditions of the belfry show some discrepancies with those specified for the NBCC response evaluation procedure. Firstly, the NBCC procedure for wind-induced responses is proposed for typical types of prismatic buildings which stand freely on the ground without obstacles. The belfry, partly, has a spire on the top and is partly connected with main hall. Also, the belfry stands on a hill and



(a) *x*-direction (b) *y*-direction

Figure 7. Peak accelerations at the top of belfry (measurement height = 31.5 m).



(a) *x*-direction (b) *y*-direction

Figure 8. Comparisons of wind-induced accelerations between measurement and analyses.

has some surrounding buildings near it.

It, however, can be confirmed in Fig. 11 that the increasing tendency of the analytical accelerations approximately follows that of the measured ones irrespective of discrepancies between the analysis and the actual conditions. This implies that wind-induced responses can be reliably evaluated using analytical methods for special masonry structures such as the belfry if dynamic properties and wind-loads are properly evaluated for the response analysis.

7. Concluding Remarks

In this research, a calibration process of a FE model was performed for building structures and a historic masonry structure in serviceability vibration level. Reference dynamic properties were evaluated from field measurement accelerations to calibrate the FE model. It was investigated whether or not the responses of the structures to vibration inputs could be reliably evaluated by comparing analyzed and measured accelerations of them. Acceleration responses of the structures were measured during typhoon events at a construction sites. The acceleration responses were also evaluated for the external excitations and compared with the analyzed results.

In the process of the FE model calibration, it was found that FEMA material model seemed to be the most appropriate for material properties of deteriorated masonry structures. The FE model could be properly calibrated when the constraint condition at ground was given as fixed. For building structures, non-structural components, flexural rigidity of floor slabs, elastic modulus of in-situ concrete, etc. should be considered in FE model construction.

It could also be observed that the analyzed acceleration responses were in a proper agreement with the measured responses to wind excitations. This implies that structural responses in serviceability level for special types of masonry structures could be reliably expected if proper FE models are achieved through proposed FE model calibration process. The calibrated FE models and their analysis results could be used for establishing maintenance criteria for historical masonry structures.

In addition, it was concluded the calibrated FE models could be used as initial FE models for constructing a FE model of structures in ultimate vibration level at which structural damages occur, such as earthquakes. The measured data and analysis results in this research could also be used as reference data to make a plan for preservations of historical masonry buildings.

References

Carini, A. and Genna, G. (2012). Stability and strength of old masonry vaults under compressive longitudinal loads: Engineering analyses of a case study. *Engineering Structures*,

40, pp. 218~229.

Applied Technology Council. (1998). FEMA 306: Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings, Federal Emergency Management Agency (FEMA).

Brincker, R., Ventura, C. E., and Andersen, P. (2001). Damping estimation by frequency domain decomposition. *Proceedings of 19th International Modal Analysis Conference (IMAC)*, 4359(1), pp. 698~703.

Brincker, R., Zhang, L., and Andersen, P. (2000). Modal identification from ambient responses using frequency domain decomposition. *Proceedings of the 18th International Modal Analysis Conference (IMAC)*, San Antonio, Texas, 4062(2), pp. 625~630.

Cagnan, Z. (2012). Numerical models for the seismic assessment of St. Nicholas Cathedral, Cyprus, *Soil Dynamics and Earthquake Engineering* 39, pp. 50~60.

Wu, C., Hao, H., and Lu, Y. (2005). Dynamic response and damage analysis of masonry structures and masonry infilled RC frames to blast ground motion. *Engineering Structures*, 27, pp. 323~333.

Fu, J. Y., Li, Q. S., Wu, J. R., Xiao, Y. Q., and Song, L.L. (2008). Field measurements of boundary layer wind characteristics and wind-induced responses of super-tall buildings. *J. Wind Eng. Ind. Aerodyn.* 96, pp. 1332~1358.

Kim, D. H. and Kim, J. Y. (2014). Assessment on Natural Frequencies of Structures using Field Measurement and FE Analysis. *International Journal of High-Rise Buildings*, 3(4), pp. 305~310.

Kim, J. Y., Yu, E., Kim, D. Y., and Kim, S. D. (2009). 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., Yu, E., Kim, D. Y., and Tamura, Y. (2011). Long-term monitoring of wind-induced responses of a large-span roof structure. *J. Wind Eng. Ind. Aerodyn.* 99, pp. 955~963.

Lourenço, P. B., Trujillo, A., Mendes, N., and Ramos, L. F. (2012). Seismic performance of the St. George of the Latins church: Lessons learned from studying masonry ruins. *Engineering Structures*, 40, pp. 501~518.

Mallardo, V., Malvezzi, R., Milani, E., and Milani, G. (2008). Seismic vulnerability of historical masonry buildings: A case study in Ferrara. *Engineering Structures*, 30, pp. 2223~2241.

NBCC. (2005). National Building Code of Canada, Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, Ont.

Park, J., Towashiraporn, P., Craig, J. I., and Goodnod, B. J. (2009). Seismic fragility analysis of low-rise unreinforced masonry structures. *Engineering Structures*, 31, pp. 125~137.

Sima, J. F., Roca, P., and Molins, C. (2011). Nonlinear response of masonry wall structures subjected to cyclic and dynamic loading. *Engineering Structures*, 33, pp. 1955~1965.