Computational modeling of a unique tower in Kuwait for structural health monitoring: Numerical investigations

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Summary
Computational modeling, in addition to data analytics, plays an important role in structural health monitoring (SHM). The high-fidelity computational model based on the design and construction information provide important dynamics information of the structure and, more importantly, can be updated against field measurements for SHM purposes such as damage detection, response prediction, and reliability assessment. In this paper, we present a unique skyscraper (Al-Hamra Tower) located in Kuwait City and its high-fidelity computational model using ETABS for structural health monitoring applications. The tower is made of cast-in-place reinforced concrete with a core of shear walls and two curved shear walls running the height of the building (approximately 413 m with 86 floors in total). Interesting static and dynamic characteristics of the tower are described. System identification, interferometry-based wave propagation analysis, and wave-based damage detection are performed using synthetic data. This work mainly presents the phase of numerical investigations, which serves as a basis for correlating the field monitoring data to the model of the building in future work.

KEYWORDS
Al-Hamra Tower, computational modeling, dynamic analysis, structural health monitoring, system identification, wave propagation

1 | INTRODUCTION

Structural health monitoring (SHM) has become a popular tool for safety and serviceability assessment of in-service civil structural systems such as buildings and bridges. Significant efforts have been made over the last two decades toward the development of sensing technologies,1,2 data processing techniques,3,4 computational modeling, and model updating5-8 as well as system identification algorithms9-11 for SHM in both lab experiments and field testing. Recently, monitoring of building structures has gained tremendous attention. For example, Snieder and Safak12 applied deconvolution-based seismic interferometry to estimate the travel waves in a tall building. Yuen and Kuok13 used the Bayesian spectral density approach with 1-year daily measurement of a 21-story reinforced concrete (RC) building to trace modal parameter variation along with environment change. Çelebi et al14 and Taciroglu et al15 investigated the rocking behavior of a concrete tall building located at the Massachusetts Institute of Technology (MIT) campus, Cambridge, MA. Recently, Sun et al16
and Mordret et al. studied the shear wave propagation characteristics of this MIT building using seismic interferometry based on ambient noises.

In addition to processing the monitoring data, computational modeling of the building also plays an important role. A finite element (FE) model can serve as a basis for optimal sensor placement analysis, model updating against sensor measurement (alternatively with condensed model), structural change (e.g., damage) quantification and localization, reliability assessment, response prediction under various simulated loading conditions, and so forth. Ni et al. presented both full-scale and reduced-order FE models of the Canton Tower (610 m), which served as a benchmark model for monitoring tall buildings in the SHM community.

This paper presents the computational modeling of the world’s tallest sculptured tower (named the Al-Hamra Tower, approximately 413 m) in Kuwait with a focus on numerical investigations in the context of SHM. This tower, made of cast-in-place RC, has a unique sculptured geometry with 86 floors. A high-fidelity full-scale FE model of the tower has been created using the commercial software ETABS (Computers and Structures, Inc.) as a baseline model for SHM of the tower and demonstration of a proposed wave-based damage detection methodology.

This paper is organized as follows. Section 2 describes the architectural and structural design information of the Al-Hamra Tower. Section 3 presents the high-fidelity computational model of the tower and its static and dynamic characteristics. In order to link the model to SHM application, Sections 4 and 5 discuss system identification of the tower using synthetic data based on modal identification, interferometric wave analysis, and wave-based damage detection. Finally, conclusions of the paper are summarized in Section 6.

2 | DESCRIPTION OF THE TOWER

The Al-Hamra tower is one of the world’s tallest sculptured towers designed by Skidmore, Owings, and Merrill LLC (see Figure 1), located at one of the most significant neighborhoods of Kuwait City. The tower is approximately 413 m high with 86 floors including three basement levels, which provides the largest office floor area in Kuwait City, with a buildup area of about 2,300 m². Construction of the tower was completed in 2011 and it has been operational since then. The tower has three major portions: the shopping mall (in the first five floors) with an extended width, the main tower with a complex lobby structure (shown in Figure 1), and multilevel car parking lots. The extended portion of the shopping mall and the multistory car park area are structurally separated from the tower. Typical floor height is about 4 m and atypical floors such as mechanical and refuge floors have double heights. The south-faced wall is made of concrete, which has numerous openings with glass windows, whereas the east-, west-, and north-faced walls are made of glass curtain walls.

FIGURE 1  The Al-Hamra Tower located in the Kuwait City
The architectural design of the tower considers the site specific environmental and urban conditions, resulting in a sculptured form. The tower is almost square in plan, but has an internal open void that is formed by cutting a slice from a typical floor slab. The southwest quadrant of the tower is removed at the base, and removed portion rotates counter-clockwise at each higher floor until the southeast quadrant is removed, as illustrated in Figure 2a. The underlying concept of this design provides a measure of environmental protection from the desert sun by a nearly solid stone façade to the south. Due to the changing (“rotating”) floor configurations, the mass center of each floor diaphragm shifts along the height. The tower was designed in such a way that the mass center of lower one-third height of the tower is offset toward east, and top one third is offset toward west with respect to the mass center the full tower. In fact, the mass

FIGURE 2  (a) Geometric sketch of the Al-Hamra Tower with changing floor sections (e.g., low-, mid-, and high- rise floors) and (b) shear walls (core structure) of the Al-Hamra Tower

FIGURE 3  The high-fidelity finite element model (ETABS) of the Al-Hamra Tower
center of middle one-third portion of the tower matches the overall mass center of the tower. Interestingly, the effect of the offsets cancels each other through the lower and the top one-third portions. However, verification of the designed offset will be performed after instrumentation of the tower is completed and analysis of vibration measurements is done. Furthermore, it will be also interesting to see how this type of offset affects the dynamic behavior of the tower with varying occupant load distribution.

The tower is made of cast-in-place RC with a core of shear walls and two curved shear walls running the height of the building as shown in Figure 2b. The “decorative” part of the tower shown in Figure 3 mainly consists of steel framing. A typical floor is made of conventional RC beam, column, slab, and shear wall framing system. The RC columns have squared cross-sectional areas with size lengths varying from 0.8 to 1.4 m, whereas the thickness of shear walls varies from 0.3 to 1.2 m. Composite RC columns with fully encased I-shaped steel sections are used for the first 28 stories. The concrete grade used for construction varies from C40 (high-rise) to C70 (low-rise). The tower is operated with multiple elevators. These elevators and mechanical shafts are within the core enclosed by RC shear walls as shown in Figure 2b. The foundation of the tower consists of a 4.2 m deep RC raft, which sits on a group of 289 RC piles ranging in depth from 22 to 27 m with a diameter of 1.2 m.

3 | COMPUTATIONAL MODELING OF THE TOWER

Based on the design information (e.g., structural drawings) of the Al-Hamra Tower, a high-fidelity full-scale 3D FE model was developed (shown in Figure 3) as a baseline model. When one considers to establish an FE model for analysis of tall buildings, commercial software such as ANSYS (Ansys, Inc.) \(^{27}\) and ETABS (Computers and Structures, Inc.) \(^{19,29}\) could be used. Herein, the full-scale FE model of the Al-Hamra Tower was built by ETABS given its distinctive advantage in modeling of building structures. ETABS offers friendly user interface to perform modeling, analysis, and design of buildings. The model consists of 33,360 joints, 32,438 frame elements, and 161,793 degrees of freedom (DOFs) and the “decorative” part (light steel framing) was not included in the modeling for simplification. The tower has a very complex geometry of its architecture, for example, the twisting shear walls and the complex lobby structure (see Figure 3), which has been meticulously modeled. The entire FE model is composed of frame elements (beams and columns), shell elements (shear walls), and rigid diaphragms (slabs) as shown in Figure 4. These elements are fully connected with each other. The frame element has two nodes with six DOFs at each node, whereas the shell element has four nodes with six DOFs at each node.

**FIGURE 4** Components (frames, shear walls, and slabs) of the Al-Hamra Tower finite element model
Windows within the shear walls were modeled as openings. This full-scale model is used for both static and dynamic analysis of the tower.

We first study the static deformation of the Al-Hamra Tower due to its self-weight loading. Due to the complex asymmetric geometry, a vertical load such as self weight on the tower structure will generate torsional movements because of geometric coupling between horizontal and vertical axes. Long-term deformation of the tower under self-weight loading might become even worse when taking into account the nonlinear geometric effect and concrete material shrinkage. The FE analysis result shows that a large deformation (the maximum deformation of about 12 cm, see Figure 5) is present due to self-weight loading. The analysis is based on the nonlinear $p - \Delta$ effect, which takes into account the nonlinear geometry of the tower. As the harsh environment in Kuwait changes over time (e.g., high temperature and its variation), it is unclear how this type of deformation will propagate. This will be further studied through field monitoring of the temporal deformation after the instrumentation is completed.

![FIGURE 5](image)

Static deformation (displacement magnitude) of the Al-Hamra Tower under self-weight loading condition considering the nonlinear $p - \Delta$ effect. Note that the numbers are presented in millimeters. For visualization purpose, the displacement magnitude in the figure is scaled.

| TABLE 1 | Calculated and measured modal frequencies of the Al-Hamra Tower |
|---------|-------------------------------------------------------------|
| Mode    | $p - \Delta$ FE  | No $p - \Delta$ FE  | Ambient Vibration | Discrepancy (%) |
|         | Freq. | Period | Freq. | Period | Freq. | Period | $p - \Delta$ FE | No $p - \Delta$ FE |
| 1st: NS Bending | 0.140 | 7.166 | 0.143 | 6.978 | 0.142 | 7.042 | 1.73 | 0.92 |
| 2nd: EW Bending  | 0.190 | 5.260 | 0.193 | 5.195 | 0.176 | 5.682 | 8.02 | 9.38 |
| 3rd: Torsion     | 0.285 | 3.504 | 0.294 | 3.406 | 0.307 | 3.257 | 7.04 | 4.37 |
| 4th: NS Bending  | 0.620 | 1.612 | 0.625 | 1.599 | 0.607 | 1.647 | 2.20 | 3.03 |
| 5th: EW Bending  | 0.736 | 1.358 | 0.740 | 1.351 | 0.658 | 1.520 | 11.91 | 12.49 |
| 6th: Torsion     | 0.815 | 1.227 | 0.821 | 1.218 | 0.835 | 1.198 | 2.40 | 1.67 |
| 7th: NS Bending  | 1.381 | 0.724 | 1.385 | 0.722 | 1.237 | 0.808 | 11.66 | 11.97 |
| 8th: Torsion     | 1.425 | 0.702 | 1.431 | 0.699 | 1.300 | 0.769 | 9.58 | 10.05 |

Note: EW: east-west; FE: finite element; NS: north-south. The units of frequencies and periods are Hertz and seconds, respectively.
Based on the full-scale FE model, the modal frequencies and mode shapes of the tower are calculated using eigenanalysis as shown in Table 1 and Figure 6. It appears that the weak direction of the tower is north-south (NS) and the strong direction is east-west (EW), depicted in Figure 3. It can be seen from Table 1 that the nonlinear $p - \Delta$ effect slightly reduces the natural frequencies making the tower more flexible. Due to the complex geometry, bending modes are heavily coupled with torsional modes, especially for higher modes. To evaluate the accuracy of the modeling, we compare the model eigenfrequencies with those obtained from a duration of 36-hr field measurements recorded by an accelerometer (3-component Kinematics Episensor and Q330 data loggers) at the top terrace of the tower temporarily deployed by researchers of the Kuwait Institute for Scientific Research. The power spectra density function of the ambient vibration measurement is calculated (e.g., the time domain signals are first divided into 10-min windows with 25% overlap and averaging is applied to all the power spectra density windows), as shown in Figure 7, where the resonance peaks of the ambient vibration spectra can be clearly seen. It can be observed in Table 1 that the calculated modal frequencies from the FE model and the ambient measurements are, in general, in good agreement, indicating an acceptable accuracy in modeling. Nevertheless, the discrepancies can be minimized through model updating based on field measurements after the full instrumentation is completed.

**FIGURE 6**  The first eight modes of the Al-Hamra Tower obtained from its finite element model considering the nonlinear $p - \Delta$ effect. The weak direction of the tower is north-south (NS) and the strong direction is east-west (EW)

**FIGURE 7**  The power spectra density (PSD) function of the ambient vibration measurement of the Al-Hamra Tower. To compute the PSD, the time domain signals are divided into 10-min windows with 25% overlap and averaging is applied to the PSD windows.
To study the dynamic characteristics of the Al-Hamra Tower in the context of SHM, system identification was performed on the synthetic data in this section. A blind source separation (BSS) technique was then applied to identify the frequencies and mode shapes.

**FIGURE 8** Sensor locations at a typical floor, represented by red and blue arrows. Here, $a_1$, $a_2$, $a_3$, and $a_4$ denote the measured accelerations, whereas $u_o$, $v_o$, and $\theta_o$ denote the condensed accelerations at the reference point $O = (0, 0)$.

**FIGURE 9** Condensed synthetic acceleration time histories recorded at every 10 stories for east-west, north-south, and torsional directions. The magnitudes are scaled by multiplying a scaling coefficient for visualization. The nonlinear $p - \Delta$ effect has been considered in the response.
4.1 Synthetic data generation

The full-scale FE model was used to generate the synthetic response of the tower subjected to ground motion input. A duration of 160-s 1999 Chi-Chi earthquake records (with scaled peak ground accelerations (PGAs) close to 0.1 g for the NS direction and 0.07 g for the EW direction) with a sampling rate of 100 Hz were input to the NS and EW directions at the basement level of the model. A modal damping was adopted with identical damping ratios of 2% for all the modes. An implicit Newmark-\(\beta\) method was used to integrate the dynamic response, where the nonlinearity due to \(p - \Delta\) effect was solved by Newton-Raphson iterations. Note that nonlinearities due to material damage were not considered in the dynamic analysis. The acceleration time histories at selected stories (stories installed with sensors) are output as synthetic data. The ongoing instrumentation design suggests that the second and the third basement levels, the sixth floor, the 16th floor, the 29th floor, the 42nd floor, the 54th floor, the 65th floor, and the 76th floor are installed with accelerometers. The sensor locations were designed mainly based on optimal sensor placement (OSP) analysis although considering physical constraints. The OSP problem was formulated as a constrained integer optimization process solved by a panelized discrete artificial bee colony algorithm. Note that if some certain floors prohibit sensor installation, the suggested sensor location by OSP was then moved upstairs/downstairs or to the nearest mechanical floor. Each floor has more than four accelerometers in order to capture the torsional response. To test the effect of measurement noise, the synthetic data is polluted with noise by adding a zero mean Gaussian white noise sequence, whose root-mean-square (RMS) is a certain percentage of the noise-free data RMS, to the noise-free data. The synthetic data (with 10% RMS noise) are then used for system identification and wave analysis in the following sections.

4.2 Story measurement condensation

The accelerometers are installed on the ceilings of selected floors with the locations at a typical floor as shown in Figure 8. For each floor, the measured accelerations can be condensed into the story accelerations in three directions at a point of interest (e.g., \(u_o\) for EW direction, \(v_o\) for NS direction, and \(\theta_o\) for torsional direction as shown in Figure 8), which are

![FIGURE 10](#) Condensed synthetic displacement time histories recorded at every 10 stories for east-west, north-south, and torsional directions. The magnitudes are scaled by multiplying a scaling coefficient for visualization. The nonlinear \(p - \Delta\) effect has been considered in the response
computed by solving the following equations using a least squares approach:

\begin{align}
\alpha_1 &= v_0 - \theta_0 y_1, \\
\alpha_2 &= v_0 - \theta_0 y_2, \\
\alpha_3 &= u_0 + \Theta x_3, \\
\alpha_4 &= u_0 + \Theta x_4,
\end{align}

(1)

where $\alpha_1$ and $\alpha_2$ are the measured accelerations along the EW direction; $\alpha_3$ and $\alpha_4$ are the measured accelerations along the NS direction; $x_3, x_4, y_1,$ and $y_2$ are the sensor coordinates in the $x-O-y$ coordinate system with $O = (0, 0)$ as shown in Figure 8, in which the coordinates of each sensor with respect to the selected reference point $O$ are given as well. Figure 9 shows the condensed synthetic acceleration time histories recorded at every 10 stories for EW, NS, and torsional directions for the case of considering the nonlinear $p - \Delta$ effect. The corresponding displacement time histories are shown in Figure 10. Because there is no explicit torsional input to the model of the tower, the condensed torsional motion at the base level is quite small as shown in Figures 9c and 10c. Nevertheless, the torsional response of the tower is distinctive because of the translational-torsional mode coupling. Figure 11 shows the roof displacement time histories with and without the nonlinear $p - \Delta$ effect for EW, NS, and torsional directions. It can be observed that the $p - \Delta$ effect is very distinctive and significantly changes the long-period motion of the tower. The condensed time histories are used for modal identification in Section 4.3 as well as wave propagation analysis in Section 5.

4.3 | BSS for modal identification

BSS aims to recover statistically independent source signals from the recorded data, which has proven to be a successful approach for modal identification given output-only measurements of structural systems.\textsuperscript{30-33} Consider a $s$-DOF

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure11.png}
\caption{Condensed synthetic displacement time histories with/without $p - \Delta$ effect at the roof level for east-west, north-south, and torsional directions}
\end{figure}
time-invariant linear system with the equation of motion given by

$$M \ddot{x}(t) + D \dot{x}(t) + Kx(t) = f(t), \quad (2)$$

where $x(t) \in \mathbb{R}^{s \times 1}$, $\dot{x}(t) \in \mathbb{R}^{s \times 1}$, and $\ddot{x}(t) \in \mathbb{R}^{s \times 1}$ are the system’s displacement, velocity, and acceleration responses; $M \in \mathbb{R}^{s \times s}$, $D \in \mathbb{R}^{s \times s}$, and $K \in \mathbb{R}^{s \times s}$ are the mass, damping, and stiffness matrices; and $f \in \mathbb{R}^{s \times 1}$ is the input. The system response (e.g., the acceleration $\ddot{x}$) can be expressed in the modal space through modal expansion $\ddot{x}(t) = \Phi \ddot{q}(t)$, where $\Phi \in \mathbb{R}^{s \times s}$ is the modal transformation matrix consisting of real-valued mode shapes and $\ddot{q} \in \mathbb{R}^{s \times 1}$ is the modal acceleration response in the modal space. It is noted that the above equation has a similar expression as compared with the basic BSS decomposition equation given by $y(t) = As(t)$, where $y(t) \in \mathbb{R}^{m \times 1}$ is a vector consisting of $m$ measured signals, $s(t) \in \mathbb{R}^{n \times 1}$ is a vector containing $n$ independent sources ($m \geq n$), and $A \in \mathbb{R}^{m \times n}$ is the mixing matrix with full rank. The objective of BSS is to simultaneously estimate $A$ and $s(t)$ from $y(t)$. For structural modal identification, BSS estimates the mode shape matrix, $\hat{\Phi} \in \mathbb{R}^{m \times n}$, and the modal response, $\hat{\ddot{q}} \in \mathbb{R}^{n \times 1}$, from the measured structural response such as accelerations, $\ddot{x} \in \mathbb{R}^{m \times 1}$.

A second-order blind identification approach was used to identify the modal quantities given acceleration measurements in this study. Mode shapes were directly extracted from the mixing matrix, whereas frequencies can be easily obtained through peak picking of the Fourier amplitude spectrum of each identified modal response. Using the condensed synthetic data with 10% RMS noise, the identified first eight mode shapes are given in Figure 12, corresponding to the identified frequencies of $0.142 (0.138), 0.181 (0.180), 0.291 (0.270), 0.607 (0.604), 0.723 (0.722), 0.788 (0.784), 1.333 (1.332)$, and $1.401 (1.397)$ Hz, where the frequencies in the parentheses represent the case of considering the nonlinear $p - \Delta$ effect. In general, the identified modal frequencies agree well with the calculated ones based on FE model as depicted in Table 1, indicating a successful identification given the designed sensor layout in the ongoing instrumentation. Nevertheless, small discrepancies can be observed between the identified frequencies and the ground truth values due to (a) the measurement noise and (b) the condensation process. The mode shapes can be well-decoupled to bending and torsional components. It is noted that, as shown in Figure 12, the identified mode shapes for both cases of considering and not considering the nonlinear $p - \Delta$ effect are almost identical despite the identified frequencies are different. Nevertheless, another interesting observation is that the nonlinear $p - \Delta$ effect has significant influence on the identified modal

![Figure 12](image-url)
responses (see Figure 13), especially for the first few modes (e.g., the first three modes). Modal responses of higher modes tend to be less affected by the nonlinear $p - \Delta$ effect. To sum up, the nonlinear $p - \Delta$ effect changes the natural frequencies and modal responses of the tower, more distinctly for lower modes, and doesn’t (or very slightly) affect the mode shapes.

5 | WAVE PROPAGATION ANALYSIS

We also performed wave propagation analysis of the Al-Hamra Tower with the synthetic data using a seismic interferometry approach.12,16,35-38 In general, the vibration of a building is related to the excitation, the soil-structure interaction, and the building mechanical properties.12 Separating the building response from the excitation and the soil-structure interaction using vibrational data yields information of the intrinsic characteristics of the building (e.g., the impulse response functions [IRFs]). We herein apply a deconvolution interferometry method to extract the shear/bending waves propagating in the building.12,16 The IRF represents the representative building response to an input delta function at the reference level, which contains the wave propagation information. With a reference measurement, the IRFs of the building can be computed as follows16,36:

$$S(z, t) \approx \sum_{n=1}^{N} \left\{ F^{-1} \left\{ \frac{y_n(z, \omega) y^*_n(z_0, \omega)}{|y_n(z_0, \omega)|^2 + \delta^2 |y_n(z_0, \omega)|^2} \right\} \right\},$$

where $S(z, t)$ is the IRF at $z$ (the floor level, $z \leq H$, with $H$ being the building height); $y_n(z, \omega)$ is the $n$th wavefield at $z$ in the frequency domain; $\langle |y_n|^2 \rangle$ is the average power spectrum of $y_n$; $\omega$ is the angular frequency; $t$ is the time; $z_0$ is the reference level.

**FIGURE 13**  Identified first eight modal responses of the Al-Hamra Tower using synthetic acceleration time histories at selected stories with/without nonlinear $p - \Delta$ effect
level of the building (e.g., the ground or the basement); \( \ast \) denotes the complex conjugate; \( N \) is the total number of time intervals of the measurements; \( \delta \) is a stabilizing parameter (water level), e.g., \( \delta = 0.002 \) in this study; and \( F^{-1} \) denotes the inverse Fourier transform. When earthquake records are used, the IRFs can be directly extracted using Equation 3 by treating the ground motion measurement as the source.

With the synthetic data described in Section 4.1, we took the condensed acceleration response at the top level of the tower as the reference source and applied the deconvolution interferometry to other condensed story measurements. The deconvolved waveforms (6 Hz low-pass filtered) for EW and NS directions are given in Figure 14. Both waveforms shown in Figure 14 contain up-going (acausal) and down-going (casual) waves. It is interesting to observe many "wiggle waves" in both the acausal and casual parts. As shown in Figure 14b, we also see that the wiggle waves are more distinctive in the weak NS direction, which might be caused by the complex geometry of the building (e.g., the changing floor sections and the twisting shear walls). It will be interesting to validate this finding from future field measurement data.

After picking the wave travel times and distances as marked by arrows in Figure 14, linear regression was performed to calculate the wave speeds for both the up-going and down-going waves. Figure 15 shows the wave speeds for both EW and NS directions.
and NS directions. It is noted that the waves below and above the 29th floor (at the elevation of about 150 m) have different propagation speeds, in which regression was performed bilinearly. The wave paths shown in Figure 15 sketch that a wave pulse starts from Upward Path 1, propagates through Upward Path 2, is reflected at the roof level, goes through Downward Path 3, and travels to Downward Path 4. The wave speeds along the wave path 1-2-3-4 are 692, 592, 588, 714 m/s for the EW direction and 667, 467, 482, 558 m/s for the NS direction. The waves propagate faster within Paths 1 and 4 compared with Paths 2 and 3. It is also seen that the up-going wave speed is different from that of the down-going wave, which might be due to attenuation effects. The wave speed for the EW direction is larger that the NS wave speed, indicating that the EW is the strong direction. This observation also matches the modal frequencies presented in Table 1 (e.g., the fundamental mode is in the NS direction).

5.1 Wave-based damage detection

The wave speed can be used as an index for structural change (e.g., damage) detection. A proof-of-concept study was performed through introducing artificial damage to the model. We reduced the shear wall resistance capacity of the first floor by 50% in the model (by multiplying a reduction coefficient of 0.5), which was considered as artificial damage analogous to earthquake-induced damage. Wave propagation analysis was conducted for the damage case with synthetic earthquake data. Wave speeds were extracted by linear regression of the travel waves for the damage case, with corresponding values listed in Figure 15. With damage of the tower, the wave speeds along the wave path 1-2-3-4 reduce to be 650, 573, 563, 660 m/s for the EW direction and 654, 440, 440, 546 m/s for the NS direction. Figure 16 shows the comparison of wave speeds between intact and damage cases. A distinctive wave speed reduction can be observed for all wave paths when damage is present. It is interesting to note that the wave speed reduction in the EW direction is relatively larger than that in the NS direction. This might be due to the fact that, at the first floor, the EW shear walls have larger contribution to the stiffness of the EW direction versus the contribution of the NS shear walls to the stiffness of the NS direction. An identical reduction (50% in this study) of the overall shear wall resistance of the first floor brings bigger impact on the stiffness reduction of the EW direction. The concept of wave-based damage detection will be further demonstrated with actual field measurements after instrumentation of the tower is completed. Note that the wave approach is also applicable to investigate the effect of environmental changes (e.g., temperature, humidity) on the temporal wave velocity variations. The environmental change induced velocity variations can be quantified via a so-called “stretching technique” and thus eliminated in structural damage detection when using wave velocity as a damage index.17 This will be addressed in future investigations during the experimental phase.

6 CONCLUSIONS

In this paper, we present a high-fidelity full-scale computational model of a unique skyscraper (Al-Hamra Tower) located in Kuwait City for structural health monitoring applications. The tower has a complex architectural geometry, which is made of cast-in-place RC with a core of shear walls and two curved shear walls running the height of the building. Complex static and dynamic characteristics of the tower are studied using the full-scale FE model. Nonlinear effect analysis shows that the complex geometry leads to a large deformation due to self-weight loading and significantly changes the seismic response of the tower. The calculated fundamental mode of the tower is about 7 s, which agrees with
the period obtained from measurement. Modal identification, vibrational wave propagation analysis, and wave-based damage detection are performed using synthetic data based on the designed sensor layout in the ongoing instrumentation. This work mainly focuses on numerical investigations, which provides a baseline model for further studies in the future after actual instrumentation is completed.

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