Identifiable stress state of wind turbine tower-foundation system based on field measurement and FE analysis

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Abstract

This research focuses on the vibration response of the wind turbine tower–foundation system to identify and describe the stress state of the system. First, the movements of an existing wind turbine tower-foundation system were measured using accelerometers at the top and middle of a 20 m tower. Elastic behaviors of the tower and anchor bolts that transfer motion to its foundation were verified using various strain gauges. The spectrum of the strain of the anchor bolts generally agreed with the acceleration spectrum. Next, three-dimensional nonlinear FE analyses were performed to identify stress states of various parts of the system. The FE model was verified through data obtained from a field-based free vibration test. Then, the model was examined with a displacing obtained by processing the field measurement data. The obtained accelerations of the tower were consistent with the measured data; however, the calculated strain of the anchor bolts was larger than the measured strain. Further analysis was conducted to predict the ultimate state at failure. The deformation and stress of the anchor bolts suggested that the anchor bolts are the dominant load-carrying mechanism and failure mode of the tower-foundation system.

Keywords: Wind turbine; Tower-foundation system; Vibration characteristic; Field measurements; Fourier spectrum; FE analysis

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1. Introduction

Wind energy is attracting attention globally because of the anticipated fast transition to a low-carbon economy, due within a century at most. The cumulative installed wind power capacity from a total of 60 countries increased to 285,761 MW by the end of 2012, with the addition of 44,951 MW in new installations [1, 2].

In Japan, wind power has become a major industry [3]. In particular, increasing wind energy production has been even more actively touted against the background of the Great East Japan earthquake and the resultant nuclear power plant accident. At the end of 2012, the total amount of installed wind (power) capacity reached 2,614 MW in Japan and wind turbine installation grew by 19% compared to the previous year. However, in the past, Japan has had troubling experiences with wind turbines during typhoons. Furthermore, with increasing demand for power generation and efficiency, the durability and long-term service life of the wind turbine tower-foundation system is critical. A guideline that requires anti-fatigue design of foundations made of reinforced concrete was enacted [4] to consider repetitive actions and their detrimental effects on the wind turbine tower-foundation system.

Clearly, appropriate verification against fatigue failure of the wind turbine foundation and understanding the external forces under complex wind conditions are needed. This study aims to capture the action of external forces to the tower and to analyze the dynamic response of wind turbines.

There have been many reports concerning the dynamic responses of wind turbines [5, 6]. However, few studies have focused on the response of the wind turbine tower-foundation system under severe environmental conditions in Japan. The authors firstly performed long-term field measurements to investigate the vibration characteristics of wind turbines by using highly sensitive accelerometers and strain gauges. In addition, three-dimensional nonlinear finite element (FE) analyses for wind turbine tower-foundation systems have been conducted using idealized static forces as input [7]. This study aims to identify the external forces that are required as input for structural analysis on the basis of FE analyses, which are most effective for assessing the wind turbine tower-foundation system. Particular emphasis is given to verifying dynamic analyses with input derived from data measured at an existing structure.

2. Field measurements

2.1. Wind turbine

The wind turbine targeted in this study was equipped in 2003 on the campus of Nihon University and along the Abukuma River in the Fukushima prefecture, Japan (Fig. 1). The wind turbine had been continuously operated for 11 years prior to the study, including during the Great East Japan Earthquake in 2011, which resulted in no visible damage. The wind turbine has a rated output and wind speed of 40 kW and 11 m/s, respectively.

The upper structure consists of a 21 m tall steel tower with a 15 m diameter rotor. The foundation made by reinforced concrete is composed by a 1.1 m thick pedestal with a 2.3 m × 2.3 m square section and 1.5-m-thick footing with a 6.5 m × 6.5 m square section. The bottom flange of the tower was tied with 64 anchor bolts to the anchor plate embedded in the foundation. This connection system of tower and foundation has been widely used for wind turbines in Japan.

2.2. Measurement conditions

Accelerometers were attached at the top and middle of the tower to capture horizontal motions of the point. These accelerometers collect 60,000 data points for 5 minutes at a sampling frequency of 200 Hz. Data were recorded every hour when the acceleration exceeded a threshold of ±0.7 m/s². Strain gauges were attached at the anchor bolts in the east, west, north, and south directions to measure the transmission of vibration from the tower to its foundation. The sampling interval and frequency were the same as those of the accelerometer.

Wind speed and direction were measured using an anemometer attached to the nacelle with a sampling frequency of 1.0 Hz. All the measurements commenced in May 2013. Structure dimensions and positions of the measuring equipment are summarized in Table 1 and Fig.2.
Table 1. Wind turbine specifications.

<table>
<thead>
<tr>
<th>Hub height (m)</th>
<th>Rotor diameter (m)</th>
<th>Rated power (kW)</th>
<th>Rated wind speed (m/s)</th>
<th>Cut-in wind speed (m/s)</th>
<th>Cut-out wind speed (m/s)</th>
<th>Rotor speed (rpm)</th>
<th>Main brake system</th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>15</td>
<td>40</td>
<td>11</td>
<td>2</td>
<td>25</td>
<td>18–69</td>
<td>Blade feathering</td>
</tr>
</tbody>
</table>

3. Analyses of measured data

3.1. Wind speed–acceleration response

Fig. 3 shows the maximum wind speed versus maximum response of acceleration. Despite the scatter of the data, shown as black dots for operating wind speeds of 2–20 m/s, the maximum acceleration increased linearly with wind speed. The red dots represent data recorded while the generator was not operating. The difference between the two data sets suggests that the blade pitch control system dampened the acceleration response. The acceleration response of the tower in the time domain is shown in Fig. 4. After the cut-in wind speed, it was confirmed that the acceleration response increased with wind speed. However, in the rated wind speed range, the acceleration and wind speed did not show correlation. The acceleration responses were rather reduced because of the mitigating effect of the operation control system, e.g., pitch control.

\[ \text{Acc} = 0.0596w - 0.1276 \]

\( w \): wind speed

\( V_c \): Cut-in wind speed

\( V_r \): Rated wind speed

Fig. 3. Max acceleration versus max wind speed in October 2013.
3.2. Response of the foundation

Fig. 5 and Fig. 6 show an example of the acceleration response and strain as a function of time, respectively, for strong wind conditions. The strain fluctuations strongly depend on the acceleration variations. The value of the response was extremely small, less than 1 μ, probably because the location of strain gauges attached to anchor bolts was not consistent with wind direction which measured max wind speed. This is something that needs be resolved.

3.3. Vibration characteristics in the frequency domain

When taking a long-term measurement, time varying character of the wind can be captured in a spectrum [8]. The natural frequency was 1.8 Hz in the primary mode and 13 Hz in the secondary mode, based on the eigenvalue analysis and free vibration tests. In most cases, the Fourier spectrum exhibited the waveform shown in Fig. 7. For the tower, several peaks were seen between the primary and secondary natural frequencies. Possibly, in the strain response spectrum, the high-frequency peaks were not observed. Therefore, it was suggested the predominant vibration mode transmitted from the tower to the foundation was the primary mode.
3.4. Displacement response of the tower

The response of the tower’s displacement was calculated by double integration of acceleration in the time domain [9, 10]. To remove noise, a digital band pass filter with pass band between about 0.1 and 30 Hz was designed [11].

The trajectory of the tower displacement is needed to understand the behavior of the wind turbine. Fig. 8 shows the trajectory of the data mentioned above. The maximum displacement was about 0.5 cm at the top of the tower in the EW direction. Furthermore, elliptical trajectories were observed for each height when the wind turbine was operating. The elliptical orbits were often observed for the tower-shaped structure with high aspect ratio owing to resonance [12]. In particular, the trajectories of the top and middle of the tower were almost similar. The elliptical vibration mode owing to resonance might induce unexpected forces to the foundation.

3.5. Phase response

A phase curve is commonly called the unwrapped phase, and can be rigorously defined in terms of the integral of its derivative, as reported by [13]. The unwrapped phase gap between the top and middle of the tower was calculated by the displacement responses in the time domain, as shown in Fig. 9. Clearly, the phases of the top and middle of the tower agreed well. Each height of the tower vibrated in the primary mode and there was no phase gap between the top and middle of the tower.

![Fig. 8.Time-domain waveform and trajectory of the displacement. Fig. 9. Unwrapped phase response of the top and middle of the tower.](image)

4. Modeling of tower-foundation structure system for FE analysis

4.1. Overview of FE model

An overview of the FE model is shown in Fig. 10. The dead weight of nacelle and blades was applied to certain elements located at the top of the tower instead of reproducing their shapes with each material density. All the members except anchor bolts and the intermediate restraining reinforcements of the pedestal were modeled by solid element. Exceptions were expressed by line element; in particular, the torque on an anchor bolt was replaced by initial strain of the lines. The boundary condition between steel and concrete was modeled by joint element based on the Mohr–Coulomb theory.

Vertical displacement was restricted at the nodes of the footing bottom surface; however, confinement of surrounding soil was not considered on the sides of the footing. Total number of nodes and elements were 48967 and 44624, respectively. Mechanical properties of constituent materials are listed in Table 2.
### 4.2. Free vibration test

The reliability of the FE model was examined by comparison of analyses and the test of the actual structure. A free vibration test was performed in a wind-free condition and under pitch control of blades; human power was used to pull the tower (Fig. 11) and initiate vibration.

Fig. 12 shows the obtained acceleration response, where the natural frequency was about 1.8 Hz in the primary mode. The structural damping calculated with Equation (1) was 0.0027. In addition,

\[
\delta = \frac{1}{m} \ln \frac{y_n}{y_{n+m}} \quad \text{and} \quad \zeta = \frac{\delta}{2\pi}
\]

where \( m \) is the number of cycles, \( y_n \) is the amplitude of the \( n \)-th cycle, \( y_{n+m} \) is the amplitude of the \( n+m \)-th cycle, and \( \zeta \) is the damping factor.

### 4.2. Verification of dynamic motion of the model

Conditions of the free vibration test were reproduced by applying a 0.8 kN horizontal force to the top of the tower. Then, the load was immediately removed and the tower was left in a free state. Horizontal displacement of the top of tower was calculated and represented in Fig. 13.

### Table 2. Mechanical properties of constituent materials.

<table>
<thead>
<tr>
<th>Unit N/mm²</th>
<th>Compressive strength</th>
<th>Tensile/yielding strength</th>
<th>Young’s modulus</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>21.0</td>
<td>1.13</td>
<td>23,500</td>
</tr>
<tr>
<td>General Steel</td>
<td>----</td>
<td>500</td>
<td>210,000</td>
</tr>
<tr>
<td>Anchor bolt</td>
<td>----</td>
<td>235</td>
<td>210,000</td>
</tr>
<tr>
<td>Rebar</td>
<td>----</td>
<td>345</td>
<td>210,000</td>
</tr>
</tbody>
</table>
The natural period derived from the analyses was 1.56 Hz, which is somewhat smaller than that obtained from the free vibration test in the field. One of the possible reasons is the mass of the electric control equipment attached inside the tower, which might not be precisely reproduced in the model. The mass of incidental facilities could be conservatively estimated at the moment of design, although the design report was the only ground of our analyses. On the basis of the test result, the structural damping factor of the analysis was set at 0.0030.

5. FE analysis

5.1. Dynamic analysis using field measurements

The processed displacement data, in two horizontal directions (Fig. 14), were input to the model by node displacement control through the top and middle of the tower. Data over 300 s were thinned out into 3000 steps to save computer memory.

The calculated accelerations of the top and middle of the tower are shown in Fig. 15. These are consistent with the accelerations directly obtained from the actual structure, which indicates that the data processing approach was appropriate for the targeted structure.

The fluctuation of strains of the anchor bolts located on the north side of the tower is illustrated in Fig. 16. The maximum amplitude was almost 10 times larger than that obtained during field measurements (Fig. 6). This discrepancy could be due to cracks or other structural weaknesses close to the anchor plate, which would reduce the transmission of stress to the anchor bolts.
5.2. Simulation of failure process by monotonic loading

5.2.1. Load carrying capacity of the system

Five monotonic loads were individually applied to the node at the top of tower by displacement control 1 mm/s to investigate the restoration of the foundation. Relationships between the moment of tower (calculated as a cantilever) and horizontal displacement at the top of tower are summarized in Fig. 17.

The tower-foundation system showed elastic response until approximately 1000 kNm corresponding to 50 mm displacement. Nonlinear behavior started after almost 1500 kNm corresponding to 50 mm displacement. After that, the residual displacements seemed to be proportional to the experienced maximum moment, although the unloading stiffness was unchanged. The stress of an anchor bolt at the north side versus displacement at the top of the tower is shown in Fig. 18. The anchor bolt reached a yielding point when the displacement at the top of tower was 30 mm. However, the yielding of one anchor bolt did not lead to failure of the tower-foundation system.
5.2.2. Deformation versus degree of pulling

Deformations of the tower-foundation system at horizontal displacements of 50 mm and 100 mm are shown in Figure 19. The steel-made tower behaved as an elastic cantilever with a stable foundation. The degree of pulling the anchor bolts differed between 50 mm (Fig. 19a) and 100 mm (Fig. 19b), although the degree of pulling was expected to be the dominant factor affecting the deformation of the tower.

The principal strains of cross section of the foundation were examined (Fig. 20). Localized principal tensile strains just beside the anchor plate in tension were observed in both cases; however, the maximum strain at 100 mm displacement (Figure 20b) was more than three times larger than the strain at 50 mm displacement (Fig. 20a). This is likely due to the 100-mm displacement exceeding the cracking criteria and the 50-mm displacement being below the criteria.

Plasticity of the tower-foundation system was caused by the cracking beside the anchor plate, which was pulled up by the anchor bolts as the inclination of the tower increased. In contrast, deformation of the steel-made tower due to bending could not be dominant; therefore, the tower did not lose its elasticity and unloading stiffness had minimal change.

Fig. 19.(a) Magnified deformation at the time of 50 mm horizontal displacement (Disp × 100); (b) Magnified deformation at the time of 100 mm horizontal displacement (Disp × 100).

Fig. 20.(a) Principal strain of cross section at the time of 50mm horizontal displacement (Disp × 100); (b) Principal strain of cross section at the time of 100mm horizontal displacement (Disp × 100).
5.2.3. Failure mode

Principal strain was contoured in a cross section of the foundation to clarify the failure mode of the system (Fig. 21). Two localizations of principal strain can be observed at the tension side; outside the joint of pedestal and footing and inside the anchor plate at the maximum horizontal displacement of 1000 mm. Thus, failure mode of the tower-foundation system was thought to be a tensile failure of concrete.

Fig. 22 shows the contour of principal strain of same cross section was examined after compulsory pulling the top of the tower back to the original position. The localization of principal strain was appeared to another side. As a result, continuous horizontal crack was formed almost along the boundary of pedestal and footing.

Considering that actual displacement of the tower and stress/strain of anchor bolts obtained by dynamic analysis verified by field measurement data, the targeted tower-foundation system might not fail unnatural wind conditions. Further studies targeting the response to an earthquake and repetitive strong winds will be examined in future.

6. Conclusions

An existing tower-foundation system for a wind turbine was investigated by continuous field measurement and FE analysis. The way of identification of stress state of wind turbine based on field measurement and FE analysis was clarified here.

- Movements at the top and middle of the tower and strain of anchor bolts were measured for one year. The in-plane track of tower was successfully calculated using double integration method.
- A nonlinear FE model for the tower-foundation system was built for the target structure. The natural frequency of the FE model was examined by comparing with the result of a field-based free vibration test. The structural damping factor of the FE model was based on data obtained in the actual test.
- Dynamic motion for a particular strong wind day was simulated using the in-plane track of tower as input. Accelerations derived from modeling were consistent with the measured data; however, the simulated strain of anchor bolts was larger than that measured in the field.
- Damage process and failure mode of the tower-foundation system was predicted by static analyses with monotonic load. The deformation and stress of anchor bolts suggested that the degree of pulling anchor bolts is the dominant load-carrying mechanism and failure mode of the tower-foundation system.
- Consequently, the safety of the targeted wind-turbine was confirmed by field measurements and analyses.

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