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<td>Wang, Qu; Nakamura, Shozo; Okumatsu, Toshihiro; Nishikawa, Takafumi</td>
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Comprehensive Investigation on the Cause of a Critical Crack Found in a
Diagonal Member of a Steel Truss Bridge

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Abstract

Ikitsuki Bridge is a three span continuous truss bridge with a length of 800 m, which was completed in July, 1991. In December 2009, a crack which seriously damages the safety of the bridge was found in a diagonal member near an intermediate pier during an inspection for determining points to be annually inspected. In order to identify its cause, some tests on the material used for the cracked member were conducted, and a long-term monitoring of wind and vibration of some diagonal members with similar structural characteristics to the cracked member had been carried out since December 2011. In this article, the outline of the crack is presented and the main causes of the crack are discussed based on the results of the tests and monitoring. The conclusions can be summarized as follows: i) The crack initiated and propagated to become approximately 200 mm long as a fatigue crack. The quality of material and weld satisfies the requirement for them. ii) The vibration of the diagonal members is induced by the wind with the velocity of 6-8 m/s and higher than 15 m/s in the direction approximately normal to the bridge longitudinal axis. The maximum stress range induced by the vibration due to 6-8 m/s wind is approximately 30-40 MPa, while that due to the wind blowing at the velocity of over 15 m/s can be 195 MPa. iii) These wind-induced vibrations are thought to be the main cause of the fatigue crack.

Key words: steel truss bridge; fatigue failure; stress range; wind-induced vibration

1 Introduction
In December 2009, a critical crack was found in a diagonal member of a three-span continuous truss bridge, Ikitsuki Bridge, near an intermediate support during an inspection for determining points to be annually inspected. In order to identify the failure mode of the crack, a fracture surface observation was conducted. As a result of the observation, it was concluded that the crack was initiated and propagated as fatigue crack.

Fatigue is the progressive and localized structural damage that occurs due to repeated or fluctuating loading. It is often a major problem limiting the load-carrying capacity and the residual life of existing structures. The correct identification of fatigue-prone details in a bridge, along with well-planned inspection routines and successful strengthening and repair schedules, can guarantee the continuous and satisfactory performance of bridges during their service life [1]. Fatigue crack starts as a very small fissure and results from cyclic stresses that are below the ultimate tensile stress, or even the yield strength of the material [2]. The brittle fracture is also possible to occur suddenly from fatigue crack without any warning, which seriously damage the safety of the bridge. Wilhelm Albert devised a test machine for conveyor chains used in the Clausthal Mines and published the first article on fatigue in 1837 [3]. After that, more and more researchers and engineers have focused on the fatigue problem.

There were more than 200 fatigue and fracture cases [1, 4, 5] reported for steel and composite bridges, and their main causes are low temperature, stress concentration, vehicle load, and wind-induced vibration etc. Hasselt Bridge in
Belgium in March 1938 and King’s Bridge in Australia in July 1962 suddenly and unexpectedly collapsed in the cold weather due to brittle fracture [6]. The Silver Bridge in the U.S. collapsed in December 1967. Investigation of the wreckage showed the cause of the collapse is the failure of a single eye-bar in a suspension chain due to unstable extension or brittle fracture of two stress corrosion cracks [4]. The Sungsoo Grand Bridge in Korea collapsed in October 1994. The collapse was triggered by fatigue failure of the vertical pin-connected hanger at the north end of the suspended truss, which could be attributed to poor construction mainly related to insufficient weld penetration, and poor maintenance of the bridge under overloaded truck traffic [7, 8].

Wind-induced fatigue of flexible structures is one of the important limit-state responses for structural design consideration [9]. Wind-induced vibrations have already caused fatigue failures in some steel bridges. In early time, some fatigue cracks by the wind-induced vibration were found in the intermediate stiffener ends of I-480 Cuyahoga River Bridge in April 1973 in the U.S., the cantilever truss vertical members of Commodore John J. Barry Bridge in March 1973 in the U.S., and the gusset ends at the diagonal member connections of Shitoku Bridge in July 1967 in Japan [5]. Although rapid progress of weld and detection technology has been made in recent years with the development of economic, unfortunately, a number of localized failures still occurred in components of steel bridges. For instance, fatigue crack was found on specific steel components and joints of a railway bridge over the Elbe River in Germany in 2000. Cyclic loads introduced in the hangers by
wind-induced bending vibrations have caused significant damage involving fatigue cracking at joint welds in railway bridge hangers [10]. Fatigue crack was also found at the panel point of the diagonal member of a through-type warren truss bridge in Japan in 2007, which was caused by the wind-induced vibration of the diagonal member [11]. In addition, based on full-scale measurement of a traffic signal support structure, Zuo [12] presented an interpretation of the data collected by the monitoring system and attempted to identify the excitation mechanisms that induced the large-amplitude vibrations. Wieghaus [13] proposed a probabilistic framework to estimate the fatigue life of a lightweight, wind-excited steel structures, and compared resulting wind-induced fatigue life distributions with compiled inspection records for a large traffic signal structure population. Hong [14] and Hosch [15] observed the fatigue failure of sign, luminaire, and traffic signal support structures caused by natural wind-induced vibration and developed a comprehensive approach for fatigue design. Alduse proposed [16] a Bayesian approach to estimate the wind-induced fatigue damage of long-span bridges while considering the uncertainties in the probability model and the parameter. Repetto [17] proposed a novel approach of two levels of formulae to evaluate the alongwind-induced fatigue of slender structures and structural elements.

As described above, the cause of fatigue crack is generally heavy live load or wind-induced vibration such as vortex-induced vibration. Since the bridge site is a strait between rural islands, the traffic condition on the bridge is not severe and strong wind often blows, the wind-induced vibration was thought to be the main cause of the
crack. The bridge has many members with structural and aerodynamic characteristics similar to the cracked member. Therefore, confirmation of the cause of the crack is very important to determine the appropriate countermeasure for crack prevention in other members. From this point of view, comprehensive investigation including material tests and a long-term monitoring of wind and vibration of some diagonal members with similar structural characteristics to the cracked member are carried out. This paper presents the outline of the investigation and discusses the main cause of the crack.

2 Outline of bridge and crack

2.1 Bridge

The bridge in which the crack was found is a three span continuous truss bridge, Ikitsuki Bridge with the main span length of 400 m, and the span arrangement is 200 m + 400 m + 200 m. It locates in the Hirado city, northwest of Nagasaki Prefecture, and connects Ikitsuki Island and Hirado Island, as shown in Fig. 1. It was completed in July, 1991. The completion of this bridge set a new world record for the main span length of this type. The photo of the bridge is shown in Fig. 2.

The main truss spacing is 13.5 m, and the effective width of the deck is 6.5 m. There are 84 kinds of diagonal members of which the shortest one is 13.24 m long and the longest one is 23.655 m long. General layout of the bridge is shown in Fig. 3.

2.2 Crack
Large crack was found in a diagonal member shown in Fig. 4 near the north side of the intermediate support (P6) during an inspection in December 2009. As shown in Table 1, the diagonal member has a rectangular box section with the width of flange and web is 500 mm and 574 mm, respectively. The thickness of plates is 9 mm except the portion near the joints where the thickness is 12 mm. The crack on sea-side flange has the length of 465 mm and that on upper web has the length of 510 mm, as shown in Fig. 5. The crack propagated along the weld toe on the outside face of the flange. The photos of the crack are shown in Fig. 6.

3 Failure analysis

3.1 Fractographic examination of the cracked component

The fracture surface of the crack is shown in Fig. 7, and the macro fracture surface of 9 mm steel plate of the part surrounded by the red line in Fig. 7 after removing the rust by acidic solvent is shown in Fig. 8. No typical characteristics of any failure modes such as chevron pattern and beach marks were observed in the macroscopic observation. The crack face in about 200 mm from upper web is perpendicular to the surface, while other part is inclined, which suggests the difference in failure mode for these parts.

A scanning electron microscope (SEM) was used to observe the crack in detail. Ductile fracture surface characterized by dimples (as shown in Fig. 9) was observed in Place A. Fatigue fracture surface characterized by striations (as shown in Fig. 10) were observed in Place B and D. The arrow in Fig. 10 indicates crack propagation
direction. In Places C and E, brittle fracture surface (as shown in Fig. 11) were observed. The observation result is summarized in Fig. 12. From these observation results, it can be thought that the crack initiated at the weld toe on the flange near the intersection to the upper web (Place A), and propagated inward and widthwise as fatigue crack up to approximately 200 mm, and then a brittle fracture occurred.

3.2 Metallographic investigation of the base material and the welds

Macrostructure were evaluated using three transverse metallographic cross-sections taken from the sea-side flange as shown in Fig. 13 (a). Fig. 14 demonstrates the macrostructures of each location. No weld defects such as incomplete fusion, blowhole and weld crack can be observed. Since the inner side structures of Figs. 14(a) and (b) are reheated structures, the joint is welded in the order from inside to outside. On the other hand, the deposited metal (Depo) remains at inner side and heat-affected zone (HAZ) is observed at outer side in Fig. 14(c). It indicates that this portion was welded again from inner side possibly for the repair of weld defect. A small fatigue crack propagated from Ikitsuki Island side was observed in Fig. 14(a). The microstructures at the crack initiation area were also investigated, and no coarse inclusions and oxide as well as abnormal structures were found.

3.3 Material testing

3.3.1 Chemical composition

Chemical analysis results of base metals for sea-side flange are shown in Table
with the specified value for SS41 steel in Japanese Industrial Standards (JIS). The values of carbon equivalent \((C_{eq})\) and cracking parameter of material \((P_{CM})\) which are indices of weld ability were calculated and listed in the table. All values fulfill the requirement of JIS [18].

### 3.3.2 Tensile test

Tensile tests on samples taken from the cracked part of sea-side flange (A1~A3, B1~B3) and road-side flange (C1~C3) as shown in Figs. 13 (a) and (b), respectively, were performed according to the JIS. The results are listed in Table 3. It shows the values of yield stress and tensile strength fulfill the requirement of JIS [18], and the strength of welds is higher than that of the base material.

### 3.3.3 Charpy impact test

Charpy V-notch impact tests were performed at 20 °C intervals from -20 °C to 20 °C. The places where the specimens were taken and shape of test specimen are shown in Fig. 13 and Fig. 15, respectively. The relationship of the absorbed energy, percent brittle fracture and temperature obtained by the Charpy impact test is shown in Fig. 16. The absorbed energy at -20 °C is more than the half of that at 20 °C, and percent brittle fracture is less than 50 %. Therefore, the transition temperature is thought to be less than -20 °C. Furthermore, the toughness of the HAZ in the welds of flange is higher than the base materials.

### 3.3.4 Vickers hardness test
Vickers hardness tests of the specimens taken from the places shown in Fig. 13 (b) were carried out for the three lines shown in Fig. 17, and the hardness distributions in the welds and the adjacent base materials (9 mm and 12mm steel plates) are shown for the weld end and central in Figs. 18 (a) and (b), respectively. The measured hardness values for the unaffected base material ranged between 150 HV and 170 HV, and those of the Depo and HAZ are approximately 20 HV higher than the base material. There is no obvious difference in hardness between the welds and base materials.

3.4 Results of material investigations

The fractographic examination revealed that the crack initiated and propagated to become approximately 200 mm long as a fatigue crack. The results of the metallographic investigation, chemical analysis, and tensile tests demonstrated that the base material and welds used in Ikitsuki Bridge satisfied the technical requirements specified in the JIS, and no defect was found in the welds. The hardness values for the Depo, the HAZ and the adjacent base materials did not reveal any inadmissible hardness peaks.

In general, the possible cause of fatigue is vehicle load or wind-induced vibration in the case of bridge. According to vehicle investigation, average daily traffic on Ikitsuki Bridge is about 3000 vehicles and no heavily traffic condition occurs on the bridge. On the other hand, the strong wind often blows in the direction normal to the bridge axis since the bridge is located on a strait between rural islands.
Therefore, the wind-induced vibration was thought to be the main cause inducing the fatigue crack.

4 Long-term monitoring

4.1 Measurement systems

In order to confirm the hypothesis on the main cause of the crack, a long term remote monitoring of wind condition and vibration of some diagonal members with similar structural characteristics to the cracked member had been carried out since December 2011. The number of windmill anemometer and strain gauge is shown in Table 4 and their observation locations are shown in Fig. 19. Wind condition had been measured by windmill anemometers at four points of each north and south side on two intermediate supports (P5 and P6). In order to reduce the influence of the main structure on the measurement of wind condition in the downstream side, the height of anemometer is set as about 5 m, as shown in Fig. 20(a). Strain gauges were set near the weld toe on two members shown in Fig. 19, as shown in Fig. 20(b) and Fig. 20(c), respectively.

The sampling frequencies of acceleration and strain were 100 Hz, while those of wind velocity and wind direction were 1 Hz. All the data were stored into files every 10 min. The real-time data were transferred and saved to the computers in Nagasaki University through the Internet.

4.2 Dynamic characteristics of measured members
The natural frequencies and the damping constants were measured by impact tests. As shown in Table 5, the 1st natural frequency of the target members in the side spans and one in the center span are approximately 6.3 Hz and 6.7 Hz, respectively. Comparing the members in the center span and those in the side spans, the frequencies of the members in the center span are higher than those in the side span since the section of the member in the center span is larger, and frequency of the cracked member has been slightly higher than others since it’s cracked portion was replaced with a stiffer member than the original one. The damping constants are less than 0.02 for all members in the target range of frequency.

By using the natural frequency and the Strouhal number [19] for rectangular section, the onset wind speed of vortex-induced vibration can be estimated as shown in Table 6. They are more than 25 m/s for all members.

**4.3 Results and discussions**

**4.3.1 Daily maximum stress range**

The stress range histogram was obtained from the time history of strain record by the rain flow counting method. Fig. 21 shows the transition of daily maximum stress range at gauge IN-7 on the diagonal member above P5 and gauge OS-7 on the diagonal member above P6. Since the fatigue strength of the target joint, i.e. as welded butt joint, is categorized into Class D in Fatigue Design Recommendations for Steel Highway Bridges [20] and its cut-off limit for variable amplitude stress is 39 MPa, certain level of fatigue damage must be accumulated. The recorded maximum
stress range is 80 MPa in the P5 member, and those in the P6 member in 2011 and 2012 is 195 MPa and 162 MPa, respectively.

In Fig. 21, threshold of 30 MPa is thought to be an index to judge the occurrence of vibration. Fig. 22 shows the number of days when the maximum stress range is over 30 MPa. The number of days with maximum stress range between 30 MPa and 60 MPa on P5 is 6 days, between 60 MPa and 100 MPa is 3 days in FY2012. And those with the maximum stress range between 30 MPa and 60 MPa on P6 in FY2011 and FY2012 is 11 days and 6 days, between 60 MPa and 100 MPa is 1 day and none, and over 100 MPa is 3 days and 1 day, respectively.

4.3.2 Wind condition inducing vibration

Wind condition on the day with the daily maximum stress range over 30 MPa was analyzed, and the average wind velocity in 10 minutes was also calculated. The analysis shows that stress range more than 50 MPa was observed under the wind with the velocity higher than 15 m/s and the direction is approximately perpendicular to the bridge axis in general. The vibration was also induced by the wind with the velocity of 6-8 m/s. In this paper, vibrations in lower and higher wind velocity ranges will be called “Phenomenon A” and “Phenomenon B”, respectively. As shown in Table 6, the onset wind velocity of vortex-induced vibration estimated from the Strouhal number of the section is 26 or 28 m/s. The onset wind velocity of Phenomenon A observed in the monitoring is much lower than those velocities. The existence of Phenomenon A was suggested in some previous studies [21, 22]. However, it has never been reported
for existing structures. Although Phenomenon B can be thought as the vortex-induced vibration, the onset wind velocity is also much lower than estimated. Therefore, the mechanism of these phenomena should be studied in the future.

As a typical example of Phenomenon A, the relationship between the maximum stress range measured by IN-7 and the average wind velocity in 10 minutes at north side of P5 on October 1, 2012 is shown in Fig. 23. It can be known that vibration is induced by the wind with the velocity of 6-8 m/s and direction between north-northeast and northeast, approximately normal to the bridge axis. The induced stress range is less than or equal to 30 MPa. In all other cases of Phenomenon A, the maximum stress ranges were approximately 30-40 MPa. The contribution of this phenomenon on the fatigue damage may be small since the stress range is relatively small.

Fig. 24 shows the relationship between the maximum stress range measured by OS-7 and the average wind velocity in 10 minutes at south side of P6 on April 3, 2012. Large stress range up to 90 MPa was induced by the wind with the velocity over 15 m/s and with the direction between southwest and west, approximately normal to the bridge axis. However, no vibration occurred when the wind blew from west-northwest, along the direction close to the bridge axis even though the wind velocity was higher than 15 m/s. The maximum stress range was 162 MPa although it is not shown in Fig. 24 due to the lack of the corresponding wind data. In other cases of Phenomenon B, the maximum stress range up to 195 MPa was recorded.
### 4.3.3 Time history of stress during wind-induced vibration

An example of stress time history when Phenomenon A occurs is shown in Fig. 25(a). The series of 10 minutes data had been recorded at IN-7 on the diagonal member above P5 from 23:50, October 1, 2012. In Fig. 25(b), stress time histories at IN-5, IN-14 and IN-7 are compared for 4 seconds including the time when the maximum stress was recorded. Stress amplitude at IN-7 moderately changes in 10 minutes. Meanwhile, it is almost constant for the 4 seconds. Stress at IN-5 is quite small, and stress at IN-7 is approximately twice larger than that at IN-14. Since this ratio is the same as the ratio of distances from the neutral axis for in-plane bending of the member, it can be known that the member vibrates in the plane of truss. The phases of time histories of IN-14 and IN-7 coincide and the frequency is 6.3 Hz which is the same as the in-plane 1st natural frequency of the member. Consequently, it can be said that Phenomenon A is the relatively stable harmonic in-plane bending vibration of the member at the 1st natural frequency.

In the same way as Fig. 25, Fig. 26(a) and Fig. 26(b) show the examples of stress time history when Phenomenon B occurs. The series of 10 minutes stress data shown in the figures were recorded from 6:50, April 3, 2012 on the diagonal member above P6. Compared with Phenomenon A, stress amplitude significantly fluctuated. The duration of large stress amplitude vibration is very short. Even for 4 seconds around the time when the maximum stress occurred, the stress did not keep constant. The ratio of stress between OS-14 and OS-7 indicates that the member mainly vibrates in the plane of truss at its 1st natural frequency. The stress at OS-5 and its
phase difference from others suggest that a certain level of out-of-plane vibration was also induced.

4.3.4 Fatigue damage and life estimated

The modified Miner’s rule in which the cut-off limit is not taken into account was used to calculate the fatigue damage of the diagonal member. The fatigue strength of the joint was assumed as Class D of *Fatigue Design Recommendations for Steel Highway Bridges* [20]. The stress range histogram shown in Fig. 27 was used to calculate the fatigue damage. The histogram was obtained by applying the rain-flow method to stress at the gauge OS-7 on the diagonal member above P6 from August 2011 to December 2012. The estimated fatigue life was 57 years. It is approximately 3 times longer than the real life of 18 years. Considering that the vibration inducing fatigue damage is rare phenomenon and the fatigue strength of welded joints is widely scattered in general, the error in the fatigue life estimation may be acceptable. Therefore, it can be concluded that the main cause of the fatigue crack is the wind-induced vibration in two wind conditions.

5 Conclusions

In this study, the outline of the crack found in a diagonal member of a three span continuous steel truss bridge is presented and the main causes of the crack are discussed based on the results of some tests and monitoring. The main conclusions can be summarized as follows.
The crack initiated and propagated to become approximately 200 mm long as a fatigue crack.

The base material and welds used in the cracked member satisfies the technical requirements specified in the JIS.

The vibration of the diagonal member is induced by the wind with the velocity of 6-8 m/s and higher than 15 m/s in the direction approximately normal to the bridge longitudinal axis.

The members mainly vibrate in the plane of truss and the vibration frequencies coincide their 1st natural frequencies.

The maximum stress range induced by the vibration due to the wind with the velocity of 6-8 m/s is approximately 30-40 MPa, while that due to wind higher than 15 m/s can be 195 MPa.

Consequently, the main cause of the fatigue crack is thought to be the wind-induced vibration in two different wind velocity, especially the higher wind velocity range.

Wind induced vibration to be a cause of fatigue crack can occur for slender members with a square section in the wind velocity range lower than expected for the vortex-induced vibration. This fact should be considered in the design and maintenance of the structures with such members.

Since the vibration resulting in fatigue damage occurred not often in a year, accurate estimation of fatigue life by the data obtained in relatively short-term is difficult. The monitoring should be continued to obtain sufficient data for statistical life estimation.
The mechanism of the vibration observed in the monitoring should be also studied since the observed onset wind velocities of the vibration were much lower than expected.

Acknowledgments The long-term monitoring has been conducted with the support of Nagasaki Prefecture. The authors would like to express their sincere gratitude to the support.

References


Fig. 1 Location of Ikitsuki Bridge
Fig. 2 Photo of the bridge
Fig. 3 Overall layout drawing of the bridge (Unit: mm)
Fig. 4 Location of the cracked member
Fig. 5 Sketch of the crack
Fig. 6 Photos of the crack

(a) Sea-side flange  (b) Upper web
(a) 9 mm steel plate                                (b) 12 mm steel plate

**Fig. 7** Fracture surface
Fig. 8 Photos of macro fracture surface
Fig. 9 Fracture surface characterized by dimples (Place A)
Fig. 10 Fatigue fracture surface characterized by striations (Place B)
Fig. 11 Brittle fracture surface (Place C)
Fig. 12 Result summary of crack observation
Fig. 13 Location of each test

(a) Sea-side flange

(b) Road-side flange
(a) Weld end (G1)    (b) Weld central (G2)    (c) Weld end (G3)

Fig. 14 Macrostructure of sea-side flange
Fig. 15 Sample of Charpy impact test
Fig. 16 Ductile brittle transition curve
Fig. 17 Three lines where the Vickers hardness test was carried out.
**Fig. 18** Result of Vickers hardness test
Fig. 19 Location for observing vibration and wind condition
Fig. 20 Photo of wind and vibration measurement sensors

(a) Windmill anemometer
(b) Strain gauges on the diagonal member above P5
(c) Strain gauges on the diagonal member above P6
Fig. 21 Daily maximum stress range (Unit: MPa)
Fig. 22 Number of days with daily maximum stress range over 30 MPa
Fig. 23 Relationship between the maximum stress range and the average wind velocity in 10 minutes

(October 1, 2012)
Fig. 24 Relationship between the maximum stress range and the average wind velocity in 10 minutes (April 3, 2012)
(a) 10 minutes history at IN-7

(b) 4 seconds history at 3 gauges

**Fig. 25** Examples of stress time history of Phenomenon A
Fig. 26 Examples of stress time history of Phenomenon B

(a) 10 minutes history at OS-7
(b) 4 seconds history at 3 gauges
Fig. 27 Stress range histogram for fatigue life estimation
Table 1 Structural characteristics of members

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C<sub>eq</sub> = C + Mn/6 + Si/24

P<sub>cm</sub> = C + Mn/20 + Si/30
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<td>317 (0.2% proof stress)</td>
<td>474</td>
<td>27</td>
</tr>
<tr>
<td>SS41 (JIS)</td>
<td></td>
<td>≧ 245</td>
<td>400～510</td>
<td>≧ 21</td>
</tr>
</tbody>
</table>
Table 4 Object of measurement

<table>
<thead>
<tr>
<th>Object</th>
<th>Location</th>
<th>Number of items</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wind direction</td>
<td>Top of middle pier</td>
<td>6, 3 points in each P5 and P6</td>
</tr>
<tr>
<td>Wind velocity</td>
<td>Top of middle pier</td>
<td>6, 3 points in each P5 and P6</td>
</tr>
<tr>
<td>Stress condition</td>
<td>Diagonal member</td>
<td>11, 5 points in P5 and 6 points in P6</td>
</tr>
</tbody>
</table>
Table 5 In-plane (in the plane of truss) natural frequency (Hz) and damping constant

<table>
<thead>
<tr>
<th>Member position</th>
<th>Center span</th>
<th></th>
<th>Side span</th>
<th></th>
<th>Center span</th>
<th></th>
<th>Side span</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
<td>North</td>
<td>South</td>
</tr>
<tr>
<td>Natural 1st</td>
<td>6.72</td>
<td>6.76</td>
<td>6.21</td>
<td>6.36</td>
<td>6.83</td>
<td>6.74</td>
<td>6.29</td>
<td>6.29</td>
</tr>
<tr>
<td>2nd</td>
<td>17.51</td>
<td>17.55</td>
<td>16.61</td>
<td>16.94</td>
<td>17.86</td>
<td>17.57</td>
<td>16.78</td>
<td>16.64</td>
</tr>
<tr>
<td>Damping constant</td>
<td>0.018</td>
<td>0.018</td>
<td>0.018</td>
<td>0.017</td>
<td>0.018</td>
<td>0.017</td>
<td>0.019</td>
<td>0.019</td>
</tr>
</tbody>
</table>
Table 6 Onset wind velocity of vortex-induced vibration

<table>
<thead>
<tr>
<th>Member position</th>
<th>Center span</th>
<th>Side span</th>
</tr>
</thead>
<tbody>
<tr>
<td>D/B</td>
<td>0.845</td>
<td>0.850</td>
</tr>
<tr>
<td>Strouhal number</td>
<td>0.12</td>
<td>0.12</td>
</tr>
<tr>
<td>Onset wind speed (m/s)</td>
<td>28</td>
<td>26</td>
</tr>
</tbody>
</table>