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3 **Comprehensive Investigation on the Cause of a Critical Crack Found in a**
4 **Diagonal Member of a Steel Truss Bridge**

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23 Abstract

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

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

43 1 Introduction

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

50 Fatigue is the progressive and localized structural damage that occurs due to
51 repeated or fluctuating loading. It is often a major problem limiting the load-carrying
52 capacity and the residual life of existing structures. The correct identification of
53 fatigue-prone details in a bridge, along with well-planned inspection routines and
54 successful strengthening and repair schedules, can guarantee the continuous and
55 satisfactory performance of bridges during their service life [1]. Fatigue crack starts
56 as a very small fissure and results from cyclic stresses that are below the ultimate
57 tensile stress, or even the yield strength of the material [2]. The brittle fracture is also
58 possible to occur suddenly from fatigue crack without any warning, which seriously
59 damage the safety of the bridge. Wilhelm Albert devised a test machine for conveyor
60 chains used in the Clausthal Mines and published the first article on fatigue in 1837
61 [3]. After that, more and more researchers and engineers have focused on the fatigue
62 problem.

63 There were more than 200 fatigue and fracture cases [1, 4, 5] reported for steel
64 and composite bridges, and their main causes are low temperature, stress
65 concentration, vehicle load, and wind-induced vibration etc. Hasselt Bridge in

66 Belgium in March 1938 and King's Bridge in Australia in July 1962 suddenly and
67 unexpectedly collapsed in the cold weather due to brittle fracture [6]. The Silver
68 Bridge in the U.S. collapsed in December 1967. Investigation of the wreckage
69 showed the cause of the collapse is the failure of a single eye-bar in a suspension
70 chain due to unstable extension or brittle fracture of two stress corrosion cracks [4].
71 The Sungsoo Grand Bridge in Korea collapsed in October 1994. The collapse was
72 triggered by fatigue failure of the vertical pin-connected hanger at the north end of
73 the suspended truss, which could be attributed to poor construction mainly related to
74 insufficient weld penetration, and poor maintenance of the bridge under overloaded
75 truck traffic [7, 8].

76 Wind-induced fatigue of flexible structures is one of the important limit-state
77 responses for structural design consideration [9]. Wind-induced vibrations have
78 already caused fatigue failures in some steel bridges. In early time, some fatigue
79 cracks by the wind-induced vibration were found in the intermediate stiffener ends of
80 I-480 Cuyahoga River Bridge in April 1973 in the U.S., the cantilever truss vertical
81 members of Commodore John J. Barry Bridge in March 1973 in the U.S., and the
82 gusset ends at the diagonal member connections of Shitoku Bridge in July 1967 in
83 Japan [5]. Although rapid progress of weld and detection technology has been made
84 in recent years with the development of economic, unfortunately, a number of
85 localized failures still occurred in components of steel bridges. For instance, fatigue
86 crack was found on specific steel components and joints of a railway bridge over the
87 Elbe River in Germany in 2000. Cyclic loads introduced in the hangers by

88 wind-induced bending vibrations have caused significant damage involving fatigue
89 cracking at joint welds in railway bridge hangers [10]. Fatigue crack was also found
90 at the panel point of the diagonal member of a through-type warren truss bridge in
91 Japan in 2007, which was caused by the wind-induced vibration of the diagonal
92 member [11]. In addition, based on full-scale measurement of a traffic signal support
93 structure, Zuo [12] presented an interpretation of the data collected by the monitoring
94 system and attempted to identify the excitation mechanisms that induced the
95 large-amplitude vibrations. Wieghaus [13] proposed a probabilistic framework to
96 estimate the fatigue life of a lightweight, wind-excited steel structures, and compared
97 resulting wind-induced fatigue life distributions with compiled inspection records for
98 a large traffic signal structure population. Hong [14] and Hosch [15] observed the
99 fatigue failure of sign, luminaire, and traffic signal support structures caused by
100 natural wind-induced vibration and developed a comprehensive approach for fatigue
101 design. Alduse proposed [16] a Bayesian approach to estimate the wind-induced
102 fatigue damage of long-span bridges while considering the uncertainties in the
103 probability model and the parameter. Repetto [17] proposed a novel approach of two
104 levels of formulae to evaluate the alongwind-induced fatigue of slender structures and
105 structural elements.

106 As described above, the cause of fatigue crack is generally heavy live load or
107 wind-induced vibration such as vortex-induced vibration. Since the bridge site is a
108 strait between rural islands, the traffic condition on the bridge is not severe and strong
109 wind often blows, the wind-induced vibration was thought to be the main cause of the

110 crack. The bridge has many members with structural and aerodynamic characteristics
111 similar to the cracked member. Therefore, confirmation of the cause of the crack is
112 very important to determine the appropriate countermeasure for crack prevention in
113 other members. From this point of view, comprehensive investigation including
114 material tests and a long-term monitoring of wind and vibration of some diagonal
115 members with similar structural characteristics to the cracked member are carried out.
116 This paper presents the outline of the investigation and discusses the main cause of
117 the crack.

118 **2 Outline of bridge and crack**

119 *2.1 Bridge*

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

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

129 *2.2 Crack*

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

138 **3 Failure analysis**

139 *3.1 Fractographic examination of the cracked component*

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

147 A scanning electron microscope (SEM) was used to observe the crack in detail.
148 Ductile fracture surface characterized by dimples (as shown in **Fig. 9**) was observed
149 in Place A. Fatigue fracture surface characterized by striations (as shown in **Fig. 10**)
150 were observed in Place B and D. The arrow in **Fig. 10** indicates crack propagation

151 direction. In Places C and E, brittle fracture surface (as shown in **Fig. 11**) were
152 observed. The observation result is summarized in **Fig. 12**. From these observation
153 results, it can be thought that the crack initiated at the weld toe on the flange near the
154 intersection to the upper web (Place A), and propagated inward and widthwise as
155 fatigue crack up to approximately 200 mm, and then a brittle fracture occurred.

156 *3.2 Metallographic investigation of the base material and the welds*

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

168 *3.3 Material testing*

169 *3.3.1 Chemical composition*

170 Chemical analysis results of base metals for sea-side flange are shown in **Table**

171 **2** with the specified value for SS41 steel in Japanese Industrial Standards (JIS). The
172 values of carbon equivalent (C_{eq}) and cracking parameter of material (P_{CM}) which are
173 indices of weld ability were calculated and listed in the table. All values fulfill the
174 requirement of JIS [18].

175 3.3.2 Tensile test

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

181 3.3.3 Charpy impact test

182 Charpy V-notch impact tests were performed at 20 °C intervals from -20 °C to
183 20 °C. The places where the specimens were taken and shape of test specimen are
184 shown in **Fig. 13** and **Fig. 15**, respectively. The relationship of the absorbed energy,
185 percent brittle fracture and temperature obtained by the Charpy impact test is shown
186 in **Fig. 16**. The absorbed energy at -20 °C is more than the half of that at 20 °C, and
187 percent brittle fracture is less than 50 %. Therefore, the transition temperature is
188 thought to be less than -20 °C. Furthermore, the toughness of the HAZ in the welds of
189 flange is higher than the base materials.

190 3.3.4 Vickers hardness test

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

199 *3.4 Results of material investigations*

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

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

212 Therefore, the wind-induced vibration was thought to be the main cause inducing the
213 fatigue crack.

214 **4 Long-term monitoring**

215 *4.1 Measurement systems*

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

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

231 *4.2 Dynamic characteristics of measured members*

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

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

244 *4.3 Results and discussions*

245 *4.3.1 Daily maximum stress range*

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

253 stress range is 80 MPa in the P5 member, and those in the P6 member in 2011 and
254 2012 is 195 MPa and 162 MPa, respectively.

255 In **Fig. 21**, threshold of 30 MPa is thought to be an index to judge the occurrence
256 of vibration. **Fig. 22** shows the number of days when the maximum stress range is
257 over 30 MPa. The number of days with maximum stress range between 30 MPa and
258 60 MPa on P5 is 6 days, between 60 MPa and 100 MPa is 3 days in FY2012. And
259 those with the maximum stress range between 30 MPa and 60 MPa on P6 in FY2011
260 and FY2012 is 11 days and 6 days, between 60 MPa and 100 MPa is 1 day and none,
261 and over 100 MPa is 3 days and 1 day, respectively.

262 *4.3.2 Wind condition inducing vibration*

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

274 for existing structures. Although Phenomenon B can be thought as the vortex-induced
275 vibration, the onset wind velocity is also much lower than estimated. Therefore, the
276 mechanism of these phenomena should be studied in the future.

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

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

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296 4.3.3 Time history of stress during wind-induced vibration

297 An example of stress time history when Phenomenon A occurs is shown in **Fig.**
298 **25(a)**. The series of 10 minutes data had been recorded at IN-7 on the diagonal
299 member above P5 from 23:50, October 1, 2012. In **Fig. 25(b)**, stress time histories at
300 IN-5, IN-14 and IN-7 are compared for 4 seconds including the time when the
301 maximum stress was recorded. Stress amplitude at IN-7 moderately changes in 10
302 minutes. Meanwhile, it is almost constant for the 4 seconds. Stress at IN-5 is quite
303 small, and stress at IN-7 is approximately twice larger than that at IN-14. Since this
304 ratio is the same as the ratio of distances from the neutral axis for in-plane bending of
305 the member, it can be known that the member vibrates in the plane of truss. The
306 phases of time histories of IN-14 and IN-7 coincide and the frequency is 6.3 Hz
307 which is the same as the in-plane 1st natural frequency of the member. Consequently,
308 it can be said that Phenomenon A is the relatively stable harmonic in-plane bending
309 vibration of the member at the 1st natural frequency.

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

318 phase difference from others suggest that a certain level of out-of-plane vibration was
319 also induced.

320 **4.3.4 Fatigue damage and life estimated**

321 The modified Miner's rule in which the cut-off limit is not taken into account
322 was used to calculate the fatigue damage of the diagonal member. The fatigue
323 strength of the joint was assumed as Class D of *Fatigue Design Recommendations for*
324 *Steel Highway Bridges* [20]. The stress range histogram shown in **Fig. 27** was used to
325 calculate the fatigue damage. The histogram was obtained by applying the rain-flow
326 method to stress at the gauge OS-7 on the diagonal member above P6 from August
327 2011 to December 2012. The estimated fatigue life was 57 years. It is approximately
328 3 times longer than the real life of 18 years. Considering that the vibration inducing
329 fatigue damage is rare phenomenon and the fatigue strength of welded joints is
330 widely scattered in general, the error in the fatigue life estimation may be acceptable.
331 Therefore, it can be concluded that the main cause of the fatigue crack is the
332 wind-induced vibration in two wind conditions.

333 **5 Conclusions**

334 In this study, the outline of the crack found in a diagonal member of a three span
335 continuous steel truss bridge is presented and the main causes of the crack are
336 discussed based on the results of some tests and monitoring. The main conclusions
337 can be summarized as follows.

338 (1) The crack initiated and propagated to become approximately 200 mm long as
339 a fatigue crack.

340 (2) The base material and welds used in the cracked member satisfies the
341 technical requirements specified in the JIS.

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

345 (4) The members mainly vibrate in the plane of truss and the vibration
346 frequencies coincide their 1st natural frequencies.

347 (5) The maximum stress range induced by the vibration due to the wind with the
348 velocity of 6-8 m/s is approximately 30-40 MPa, while that due to wind higher than
349 15 m/s can be 195 MPa.

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

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

357 Since the vibration resulting in fatigue damage occurred not often in a year,
358 accurate estimation of fatigue life by the data obtained in relatively short-term is difficult.
359 The monitoring should be continued to obtain sufficient data for statistical life estimation.

360 The mechanism of the vibration observed in the monitoring should be also studied since
361 the observed onset wind velocities of the vibration were much lower than expected.

362

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366

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Fig. 1 Location of Ikitsuki Bridge

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Fig. 2 Photo of the bridge

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(a) Elevation view

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(b) Planer view

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(c) Side view

Fig. 3 Overall layout drawing of the bridge(Unit: mm)

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Fig. 4 Location of the cracked member

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Fig. 5 Sketch of the crack

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(a) Sea-side flange

(b) Upper web

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Fig. 6 Photos of the crack

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(a) 9 mm steel plate

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(b) 12 mm steel plate

Fig. 7 Fracture surface

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Fig. 8 Photos of macro fracture surface

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Fig. 9 Fracture surface characterized by dimples (Place A)

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※Arrow indicates crack propagation direction

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Fig. 10 Fatigue fracture surface characterized by striations (Place B)

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Fig. 11 Brittle fracture surface (Place C)

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Fig. 12 Result summary of crack observation

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(a) Sea-side flange

(b) Road-side flange

Fig. 13 Location of each test

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(a) Weld end (G1)

(b) Weld central (G2)

(c) Weld end (G3)

Fig. 14 Macrostructure of sea-side flange

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Fig. 15 Sample of Charpy impact test

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Fig. 16 Ductile brittle transition curve

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Fig. 17 Three lines where the Vickers hardness test was carried out

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(a) Weld end (V1)

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(b) Weld central (V2)

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Fig. 18 Result of Vickers hardness test

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Fig. 19 Location for observing vibration and wind condition

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(a) Windmill anemometer

(b) Strain gauges on the diagonal member above P5 (c) Strain gauges on the diagonal member above P6

Fig. 20 Photo of wind and vibration measurement sensors

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Fig. 21 Daily maximum stress range (Unit: MPa)

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Fig. 22 Number of days with daily maximum stress range over 30 MPa

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107 **Fig. 23** Relationship between the maximum stress range and the average wind velocity in 10 minutes
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(a) 10 minutes history at IN-7

(b) 4 seconds history at 3 gauges

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Fig. 25 Examples of stress time history of Phenomenon A

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(a) 10 minutes history at OS-7

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Fig. 26 Examples of stress time history of Phenomenon B

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Fig. 27 Stress range histogram for fatigue life estimation

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Table 1 Structural characteristics of members

Member postion		Center span	Side span
Length (m)		26.803	27.335
Flange	Section	Rectangular box	
	Width (mm)	500	500
	Thickness (mm)	9	15
	Web		
Web	Width (mm)	574	558
	Thickness (mm)	9	11
Material		JIS-SS41	

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7 $P_{cm} = C + Mn/20 + Si/30$

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Table 3 Results of tensile test

Sample	No.	Yield strength (0.2% proof stress, MPa)	Tensile strength (MPa)	Elongation (%)
Steel plate (9 mm)	A-1	286	445	42
	A-2	286	446	41
	A-3	287	446	41
Steel plate (12 mm)	B-1	294	459	43
	B-2	295	458	41
	B-3	296	459	40
Welds	C-1	304	476	24
	C-2	318 (0.2% proof stress)	477	22
	C-3	317 (0.2% proof stress)	474	27
SS41 (JIS)		≥ 245	400~510	≥ 21

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Table 4 Object of measurement

Object	Location	Number of items
Wind direction	Top of middle pier	6, 3 points in each P5 and P6
Wind velocity	Top of middle pier	6, 3 points in each P5 and P6
Stress condition	Diagonal member	11, 5 points in P5 and 6 points in P6

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16 **Table 5** In-plane (in the plane of truss) natural frequency (Hz) and damping constant

Member position		P5				P6			
		Center span		Side span		Center span		Side span	
		North	South	North	South	North	South	North	South
Natural	1 st	6.72	6.76	6.21	6.36	6.83	6.74	6.29	6.29
frequency	2 nd	17.51	17.55	16.61	16.94	17.86	17.57	16.78	16.64
Damping constant		0.018	0.018	0.018	0.017	0.018	0.017	0.019	0.019

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Table 6 Onset wind velocity of vortex-induced vibration

Member position	Center span	Side span
D/B	0.845	0.850
Strouhal number	0.12	0.12
Onset wind speed (m/s)	28	26

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