CAUSE IDENTIFICATION OF FATIGUE CRACKS IN PLATE GIRDER-ON-STEEL FRAME PIER BRIDGE

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The purpose of this study is to identify the causes of fatigue cracks found in a plate girder-on-steel frame pier bridge. The complexities of this type of structure make the stress configurations near crack locations complicated. A three-dimensional finite element model, incorporating both superstructure and substructure was created to study the stress development mechanisms near crack locations. Field tests were carried out to investigate structural behaviors, such as conditions of bridge bearings, and to validate the finite element modeling. The results indicate that fatigue cracks are caused by dysfunctional bridge bearings, poor structural details and deformations of some members.

Key Words: plate girder, steel frame pier, fatigue crack, finite element method

1. INTRODUCTION

Recently, many existing steel bridge structures were found to suffer a large number of fatigue cracks. The fatigue cracks are harmful to the structure because they shorten the service life span of the structure, and in some instances they led to brittle fracture and collapse. Because of this, the repair of fatigue damaged components is required. However, the repairs will be effective and successful only if the repair methodology directly addresses the causes of cracking. Therefore, before undertaking repairs, the causes of the cracking must be well understood. The understanding of causes of cracking also helps to detect cracks in other similar existing structures.

This paper deals with fatigue cracks found in a plate girder-on-steel frame pier bridge, which is a bridge in the inventory of the Metropolitan Expressway (MEX) of Tokyo. The bridge structure is very complex because it is located in central Tokyo; the construction constrains, such as limited land area and proximity of buildings and canals, dictate the bridge must have various curvatures and gradients. The bridge superstructure is supported by distinctive steel frame piers that are asymmetric and have irregular shape; heights and slopes differ from pier to pier. Because of these complexities, under the live load of traffic, the bridge components of both superstructure and substructure undergo complicated deformations and stress configurations. Hence, the causes of fatigue cracking, which were found in both superstructure and substructure, are difficult to interpret. Therefore, the purpose of this paper is to examine the causes of fatigue cracks in this bridge. In relation to the causes of fatigue cracks in this study, the major concern is stress characteristics at crack locations. Attention is focused on factors that may influence the mechanism of stress development at crack locations, such as structural movement and deformation.

The finite element analysis method (FEM) was used to assess the stress characteristics at crack locations and to study the influences of several parameters. In order to capture all the complicated behaviors of the bridge, a three-dimensional (3D) finite element model that incorporates both superstructure and substructure was created. More than one hundred analyses of the model were performed under various loading cases and...
Fig.1 General views of Kanda Bridge

(a) Top view

(b) Bottom view

Fig.2 Superstructure of Kanda Bridge

(c) Cross section

(d) Detail of girder ends and supports

Fig.3 Details of steel frame piers of Kanda Bridge

(a) Pier P2

(b) Pier P3
parameters. Field tests were carried out to investigate structural behaviors, such as the conditions of bridge bearings and stresses responses to a test truck. The tested results were also used to validate the finite element model. The causes of fatigue cracks are determined from the analytical and measurement results.

2. BRIDGE DESCRIPTION

(1) Bridge structure

The Kanda Bridge is a bridge in MEX that has a large number of fatigue cracks. It was selected as the object bridge in this study. The bridge was opened to traffic in 1964 and is located on the Inner Circular route of MEX. Its structure consists of multi simple span elevated steel plate girders supported by steel frame piers. Fig.1 shows the general views of this bridge. The spans vary in length from 23.4 m to 43.8 m. A total of 32 spans, having a total length 1040 m, were inspected, and in every span various members were found to have numerous fatigue cracks.

Among the 32 spans, a single span that has cracks of all types was selected as a target span for finite element modeling and field tests. Fig.2 shows the target span and includes an elevation view, a framing plan, a cross sectional view and details of girder ends and their supports. The span measures 28.6 m in length between pier P2 and P3. The superstructure is typical RC slab on steel plate girder.

It has 6 main girders, each measuring 1.8 m in depth. All girders are straight, except girder Gf, which has a slight radius of curvature. The interior girders have cut-off ends with rounded notch details, depth reducing to 1.1 m at both ends, whereas the exterior girders are of uniform depth throughout their lengths. The bridge roadway has 4 lanes, 2 lanes for each direction of travel. Fig.2(d) shows the locations where cracks were found. The cracks will be described in detail in the next section.

Fig.3 shows steel rigid frame piers used to support the superstructure of the target span. In both piers, a steel box cross beam is connected to two circular columns. One side of pier P2 is a T-shaped framed structure, which provides a cantilever cross beam section. Pier P3 has a frame structure of typical shape. The distinctive characteristic of these piers is that the bearing seats for supporting girders are not located on the tops of the cross beams: in stead, the brackets that are welded to the web plate at both sides of the cross beams serve as the bearing seats.

(2) Fatigue cracks

A large number of fatigue cracks were identified in Kanda Bridge. They could be categorized into three major types, according to locations: (a) cracks at cut-off girder ends, (b) cracks at brackets, and (c) cracks at beam-to-column connections. The cracks at each location are described as follows.

a) Cracks at cut-off girder ends (A in Fig.2(d))

As shown in Fig.4, fatigue cracks at cut-off girder ends were found at the welds of rounded notch details. The weld connection between web and flange was fillet weld. The apparent leg length of the weld was known to be 6 mm by field investigation. The cracks were classified into two types: toe cracks and root cracks. Toe cracks refer to those cracks that initiated from a fillet weld toe on a girder web plate at the center of a rounded notch detail. These cracks propagated along the weld toe and extended into the web. Fig.4(b) shows a photograph of toe crack. Root cracks refer to those

<table>
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<tr>
<th>Crack type</th>
<th>Number of crack locations / Total number of inspected girder ends</th>
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<tr>
<td></td>
<td>1978</td>
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<td>Toe cracks</td>
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Note: (*) indicates the most often found crack type.
Cracks inside box

A. Slot corner
(See Fig. 6(a))
B. Bracket upper flg.
(See Fig. (b))
C. Stiffener
D. Diaphragm

Cracks outside box

F. Bracket flg.
E. Slot corner
G. Bracket lower flg.

Note: (b): crack at weld bead; (c): crack at weld toe on cross beam lower flange; (d): crack at weld toe on diaphragm;
(f): crack at weld toe on bracket flange; (s): crack at weld toe on stiffener; (w): crack at weld toe on web plate

Fig. 5 Details of bracket and schematic drawings of cracks at brackets

(a) Crack at slot corner on weld toe A(w)
(b) Crack at bracket upper flg. on weld root B(b)

Fig. 6 Photographs of cracks at brackets and welding sections

Table 2 Summary of fatigue cracks at brackets in Kanda Bridge

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<thead>
<tr>
<th>Crack locations</th>
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<tr>
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<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A. Slot corners</td>
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<td>1</td>
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</tr>
<tr>
<td>(f)</td>
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<td></td>
</tr>
<tr>
<td>(w)</td>
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Note: (*) indicates the most often found crack location

Cracks that initiated from a weld root and propagated across the weld into the web.

Fatigue cracks at notch details have previously been identified in many steel structures\(^{1, 2}\). In Kanda Bridge, these cracks were first discovered in 1978\(^{3}\) at 22 out of 162 inspected locations of 14 spans. The most recent inspection was carried out in 2000 involved 250 girder ends. A total of 47 locations were found to have cracks (including those found earlier). In the target span, all girder ends were found to have cracks. As summarized in Table 1, the majority of cracks found in this bridge are toe cracks, whereas only a few cracks were found to be root cracks.

b) Cracks at brackets (B in Fig.2(d))

Fig. 5 shows details of fatigue cracks found in brackets. The bracket upper flanges (U-flg.) were inserted into steel box cross beams, passed through slots in the cross beam web, and fillet welded at both sides of web plates, all around the flange surfaces. Other steel plate members of a bracket, such as its web and lower flange (L-flg.), were welded to the cross beam web only outside the box. Diaphragms and stiffeners were provided inside the box.
As illustrated in Fig. 5, many different details of fatigue cracks were found at weld connections of various locations (A to G) both inside and outside the box cross beam. For each location, cracks could be further classified into cracks at weld toe of one of the base metals and cracks in the weld bead itself. However, despite these varieties of cracks, only two crack details occurred frequently: cracks at welds of slot corners (A and E) and cracks at bracket U-flg. (B and F). Among 500 brackets of the 32 spans inspected for cracks inside the box, 290 (58%) had developed cracks at slot corners or bracket U-flg, as summarized in Table 2. Moreover, among 491 brackets of the 32 spans inspected for cracks outside the box, 147 (30%) had also developed cracks at the slot corners or bracket U-flg. Investigation for cracks in target span revealed that, cracks at slot corners mostly initiated at weld toes on cross beam web plates (w), whereas cracks at bracket U-flg. mostly initiated from weld roots (b). Fig. 6 shows photographs and cross sections of cracks at these two locations.

c) Cracks at beam-to-column connections

(C in Fig. 2(d))

As shown in Fig. 7, two different types of beam-to-column connection are used in Kanda Bridge. In a Type 1 connection, the cross beam web plate was inserted into the column. The corners of the web plate (enlarged view shown in the circle) were scalloped so to allow the column web to fit in. The cross beam flange was fillet welded to the column web both inside and outside the column. In a Type 2 connection, the cross beam web and flange end at the outer surface of the column and were fillet welded both inside and outside the box.

Fatigue cracks were found only in Type 1 connections and can be classified into 2 different types, according to locations. The first crack, Crack 1, was found at weld connections between the cross beam lower flange and column web, located near the edges of the lower flange. The second crack, Crack 2, was found at weld connections directly under the cross beam web plate. Fig. 7 shows schematic drawings and photographs of these cracks. Table 3 summarizes the number of cracks found at beam-to-column connections in Kanda Bridge. Among 78 weld connections of the 32 spans, 4 had developed Crack 1. Among 156 locations of weld under web plates, 28 had developed Crack 2. Both types of cracks were also found in the target span.

Although these two cracks were found at either the weld toe of one of base metals or on the weld itself, all these cracks are likely to develop from the weld root. As shown in Fig. 8, an advanced investigation on a crack location (a different location from that shown in photograph in Fig. 7) performed by
removing the weld surface by grinding revealed the weld root with large root gap, and the crack initiated from this weld root. This large root gap can be considered as the results of the difficult welding conditions due to the scalloped details and plate alignment between the cross beam flange and the column web of the connection type 1.

3. FIELD TESTS

Field tests were carried out in order to investigate structural behaviors, such as bridge bearing displacements and responses of girders and cross beams due to test truck load.

A strain gauge was installed at the bottom surface of the lower flange of each girder, at the midspan section. The locations of these gauges in transverse direction were at halfway between the edge and center of the flange. In addition, in each pier a strain gauge was installed at the bottom surface of the lower flange of cross beam in longitudinal direction, at the midspan section. To measure movements of the bridge bearings, displacement transducers were placed at two movable bearings of girder Gd and Ge on pier P3.

A 3-axle truck weighing 245 kN (25 tons), shown in Fig.9, was then driven across the bridge without the interference of other traffic vehicles. Because the bridge was not closed to traffic, the tests were conducted at midnight in order to avoid heavy traffic volume. The tests were performed for two cases: i.e., passage of the truck in lane Out-1 and passage of the truck in lane In-1, as shown in Fig.10. For each case, the test consisted of two truck passages at a crawl speed followed by the passage at normal speed. During passage of the test truck, strain and displacement responses were collected dynamically at a sampling rate of 100 Hz. For the case of crawling-speed tests, the truck stopped at the quarter and midspan for about 5 seconds for measurement of static responses.

The measurement results of bearing movement at girder Gd and Ge indicated that the original movable bearings were completely fixed as the result of corrosion damage: the displacement transducers detected no movement. Other bearings also had no movement as inspected by naked eyes. All bearings had same severe corrosion damage as shown in Fig.11. The girder and cross beam responses are used to validate the finite element modeling and will be discussed in the next section.

4. FEM ANALYSIS

(1) Finite element model

Finite element analysis method (FEM) was used to assess stress characteristic near crack locations under various loading locations. It was also used to study effects of several parameters, such as bearing conditions and steel frame piers, which will be explained later.

In order to capture all complicated behaviors in the bridge, a fine mesh three-dimensional (3D) model of the target span including superstructure and substructure was created. Moreover, in order to take into account of the effects of the two adjacent spans, these two spans were included in the model as a simplified grid model. The entire model of three spans consists of a total of 110610 nodes and 112943 finite elements, as illustrated in Fig.12(a). The model was analyzed by a commercial finite element program so called ABAQUS.\(^4\)

The fine mesh model of the target span included the following members: for the superstructure, the RC deck, girders and its stiffeners, and diaphragms; for the substructure, the box cross beam with stiffeners and diaphragms inside, brackets, and circular columns. Figs.12(b) and (c) show the closer view of the mesh of the target span. All members
incorporated in the target span were modeled with shell elements, except for the RC deck, which was modeled with solid elements. In order to obtain local stress at a satisfactory level, elements near cracks were assigned mesh sizes as small as half the thickness (0.5t) of steel plates used for those members.

The simplified grid models of the adjacent spans were modeled by using beam elements, except that the RC slab was modeled with shell elements. The model includes RC slab, girders, diaphragms, box cross beams, and columns of the piers. The slab and girders were linked by rigid beam elements.

The material models for both steel and concrete were linearly elastic with Young’s modulus equal to 210000 and 240000 MPa, and poisson’s ratio equal to 0.3 and 0.16, respectively.

(2) Loading patterns and boundary conditions

Refer to B-live load design specifications for highway bridge⁵; the design truck axle load that consists of a pair of 98.1 kN (10t) concentration load spaced 1800 mm in the transverse direction was used in the analysis. The target span was divided into 9 sections in the longitudinal direction for designating loading locations. For each analysis, the axle load was applied on the wheel line of each lane at an individual section. Fig.13 shows an example of an axle load on lane Out-1 at section 2. Therefore, each parametric study employed a total of 36 loading cases (for 4 lanes and 9 sections).

According to the test results, the original moveable bearings were fixed by severe corrosion damage. The main FEM model used in this study, therefore, has fixed boundary condition (F-F) between girder ends and brackets. However, the model with original moveable bearings (F-M) was also analyzed and used for comparison purpose. The bases of the piers (P1 to P4) were assumed to be rigid to the ground. Therefore, the nodes representing the base of the piers were fully fixed against translation and rotation in all directions.

Fig.12 FEM model of Kanda Bridge
Fig.13 Loading sections for FEM analysis
(3) Model verification

To verify the reliability of the model, the static truck loadings of field tests were simulated with an FEM model with F-F bearing condition. The analytical results were compared with the results of the field tests.

For the substructure, Fig.14(a) and (b) compare analytical and measurement stresses at lower flanges of girders at the midspan section produced by the truck at midspan of lane Out-1 and In-1, respectively. For the substructure, Fig.15 compares analytical and measurement stresses at the lower flanges of cross beams at the midspan section produced by the truck at the quarter (L/4) and midspan (L/2) of both lanes Out-1 and In-1. Fig.15(a) shows results of pier P2, and Fig.15(b) shows results of pier P3. Both figures show very close agreement between analytical and measured results, indicating that the model simulates the bridge behaviors well.

5. ANALYTICAL RESULTS AND DISCUSSION

(1) Bearing force behaviors

The investigation of the bearing force is important, since they act directly on the girders and brackets, which are the members that develop fatigue cracks: therefore, the bearing forces can influence stress configurations near crack locations. The effects of two factors, the bearing conditions and the existence of steel frame piers, were investigated by parametric study using FEM analysis.

a) Effects of bearing conditions

The F-F bearing condition resulting from corrosion damage must make the bearing forces different from those produced under the original designed F-M condition. In order to determine the effects of bearing condition, the analytical results from these two conditions were compared.

Fig.16 compares the bearing forces of girder Gb on pier P3 produced under F-M and F-F conditions (in response of the axle load passing on the roadway). Fig.16(a) shows vertical bearing forces and Fig.16(b) shows longitudinal bearing forces.

In case of vertical bearing forces (Fig.16(a)), no appreciable difference is found between the two cases. In other words, the change in bearing conditions has no influence on vertical bearing forces. This behavior is also found in other girders.

On the other hand, the horizontal bearing forces in longitudinal direction (Fig.16(b)), which do not exist under the F-M condition, are induced under the F-F condition. Moreover, the forces are induced
even when the load is on far lane relative to the girder Gb under consideration (lane In-1 in this case). Another interesting finding is that the direction of these horizontal forces depends on the position of the load in the transverse direction. As can be seen, the forces are positive under loading on nearby lanes (Out-1 and Out-2), and negative when the load is on far lane (In-1 and In-2). A structure subjected to reverse forces is vulnerable to fatigue, since the reversal of force direction increases the stress range to which the structure is subjected. Horizontal bearing force was found to exhibit the same behavior for all other interior girders. However, for the exterior girders, Ga and Gf, the horizontal forces do not exist under either condition.

b) Effects of steel frame piers

In bridge design, bearing forces are considered to be the girder reaction forces. In general, they are calculated in consideration of only the superstructure subjected to design load: the substructure is not taken into account. However, in an actual bridge, the superstructure and substructure work together as an integral system, and the bearing forces could be different from those obtained from conventional analysis. In order to determine whether or not the existence of steel piers influences bearing forces, the analytical results obtained from a model of entire system were compared with those obtained from a model of only the superstructure. Note that both models were analyzed under F-F bearing condition.

Fig.17(a) shows the distribution of vertical bearing forces on pier P3 produced by loading on lane Out-1 at section 1, and Fig.17(b) shows the distribution of horizontal bearing forces produced by loading on lane Out-1 at section 5. Section 1 and 5 are selected because these locations exert largest vertical and horizontal bearing forces, respectively, on pier P3. For the case of vertical bearing forces, the two models can be seen to yield a difference of about 20%. For the horizontal bearing forces, the model of entire system yields forces as low as about 60% less than those yielded by the model of only the superstructure. These differences in bearing force distribution between the two models are possibly caused by the effect of the flexibility of the steel piers. This finding suggests that the effect of steel piers should be taken into account when performing analysis of bearing forces. All analyses in this study are based on the model of entire system.
(2) Stresses and cracks at cut-off girder ends

The rounded notch at cut-off girder ends is well recognized as a detail of low fatigue resistance, in view that stress concentrations develop near the notch. The two types of cracks found at the notches, previously explained, are dependent primarily on loading conditions to which the girder webs are subjected. The cracks are likely to develop from weld roots (Root cracks) if the girder webs are subjected to only in-plane load. In the case where the girders are also subjected to out-of-plane load, cracks are more likely to develop at weld toes (Toe cracks), which is frequently found to be the case with the object bridge.

These results agree with those of the inspection conducted in 1978 (at the first time of discovering these cracks), as out-of-plane deformation was found in almost every inspected girder web. At that time, stop holes of 5 mm diameter were drilled in order to remove the crack tips, and 9 to 12 mm thick steel plates were bolted on both sides of the end panels of all 162 girder webs, in an attempt to prevent out-of-plane deformations. However, this repair method was ineffective, because, as seen in Fig.4(b), the recent inspection revealed that the cracks had extended beyond the stop holes.

In addition to the out-of-plane deformation of girder webs, the frozen bearings can also cause fatigue cracks at rounded notch details. In order to explain the effect of frozen bearings, the principal stresses at a rounded notch of girder Gb on pier P3 produced by the axle load passing on the roadway are plotted in Figs.18 and 19 for the cases of F-M and F-F bearing conditions, respectively. These stresses were taken from the first node next to the notch line, which is 15 mm away from it. The principal direction, \( \phi \), is the angle between the horizontal and the maximum principal stress, \( \sigma_1 \), with the clockwise direction being positive.

In the case of the F-M condition (Fig.18), the principal directions remain nearly constant at about 40° to 42° during the passage of load throughout the span. When the load is on the nearby lane (Out-1, Fig.18(a)) the stresses are in tensile, with a maximum of 23.8 MPa, and when the load is moving on the far lane (In-1, Fig.18(b)) the notch detail is almost free from stress. Stress results from other lanes are not shown, but they fall between the results of lane Out-1 and In-1. Therefore, maximum and minimum stresses from lane Out-1 and In-1 are the upper and lower extremes of stresses that occur at girder Gb under the F-M condition. The stress range can then be calculated and found to be equal to 23.8 MPa (Max 23.8-Min 0).

Under the F-F condition, the principal directions are aligned at about 97° to 99° when the load is on the nearby lane (Out-1, Fig.19(a)). However, the
Table 4 Summary of stress ranges at notch details (MPa)

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<th>F-M</th>
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On pier P3

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</tbody>
</table>

largest stress magnitude is obtained from the minimum principal stress, which is compressive. Therefore, the dominant stresses are compressive with a magnitude of 26.3 MPa and aligned about the horizontal. Unlike the case of F-M, the notch detail is also subjected to stresses even when the load is on the far lane (In-1, Fig.19(b)). The stresses are tensile with a magnitude of 15.8 MPa and aligned at about 7° to 9°. This shows that when the load moves from lane Out-1 to In-1, the stress directions remain nearly constant and horizontal, but their signs change from compression to tension. This behavior is obviously caused by the reversal of horizontal bearing force directions described previously. The stress range at notch detail of girder Gb for the F-F condition is 42.1 MPa (Max 15.8-Min -26.3), which is much higher than that for the case of the F-M condition. Furthermore, the frequency of stress occurrence at notch details in the F-F condition is also higher, as the notches are always subjected to stresses regardless of loading position.

The stress ranges on other girders in the target span were calculated and are summarized in Table 4. The ratios between two cases indicate that for every girder, except only girder Gc on pier P3, stress ranges obtained from the F-F condition are higher than those obtained from the F-M condition. It is difficult to explain the lower stress range under F-F condition of girder Gc on pier P3 because of the complexity and asymmetry of the structure. However, one possible reason could be the effect of the column P2-2 (Fig.2(b)), which directly supports girder Gc. From the results discussed above, it can be deduced that the cut-off girder ends suffer severe fatigue damage because of the high range and high frequency of cyclic stress induced by the F-F bearing condition.

(3) Stresses and cracks at brackets

Fatigue cracks at slot corners and those at bracket upper flanges were frequently found at the brackets. These two cracks are discussed separately.

a) Cracks at slot corners

To illustrate the stress development mechanisms at slot corners, Fig.20(a) and (b) show the deformations of pier P2 (magnified by 1000 times) produced by the axle load on lane In-1 and Out-1, respectively (at section 9, directly above pier P2). The corresponding stresses induced at the corner of bracket Gf are also included in the figure. The stresses were taken from the first node next to the corner of the bracket, which is 10 mm away from the corner.

In case of loading on lane In-1 (Fig.20(a)), directly above the girder Gf, the bearing force acting on the bracket under consideration, Gf, is extremely high. In this situation, the bearing force induces high shear stresses on the cross beam web plate, in addition to the flexural stresses caused by the cross beam deformation. Subsequently, the welds at slot corners are also subjected to shear stresses and flexural stresses. It was found that the resultant principal stress at the slot corner of bracket Gf is 9.36 MPa, and its direction is about 65° counterclockwise from the horizontal.

In case where load acts on lane Out-1 (Fig.20(b)), the load is far from girder Gf, and the bearing force on bracket Gf is almost zero. However, the cross beam still undergoes deformation; consequently, the slot corner is subjected to flexural stresses in the web plate. The resultant principal stress at the slot corner of bracket Gf is a compressive stress of 5.0 MPa. Its direction is almost horizontal, because this case involves no shear stress component in vertical direction.

These results suggest that the stresses at slot corners are induced by girder reaction forces acting directly on brackets and by deformations of the cross beam. In view that the cross beams deform under all loading positions, the welds at the slot corners are always subjected to significant stresses regardless of the position of the load on the roadway.

Note that the FEM model used in this study does not incorporate the weld geometry. In an actual structure, the weld discontinuities, especially at the weld toe, produce stress concentrations. Therefore, the stress conditions in the real structure are more severe than those obtained from FEM analysis.

b) Cracks at bracket upper flanges

Fig.21 shows stress distributions in the direction perpendicular to the weld connection between bracket U-flg. and cross beam web of bracket Gf of pier P2 produced by loading on lane Out-1 and In-1. Refer to Fig.20 for loading positions and bracket Gf.
**Fig. 20** Deformation of steel frame pier P2 associated with principal stresses at slot corner of bracket Gf

**Fig. 21** Stress distribution along weld connection between bracket upper flange and cross beam web of bracket Gf on pier P2

**Fig. 22** Large root gap at upper bracket flg.

**Fig. 21(a)** shows stresses on the cross beam web, and **Fig. 21(b)** shows stresses on the bracket flange. These stresses were taken from the first nodes next to the bracket and the cross beam web connection, which are 10 mm away from the connection (indicated by the reference line). Both figures also compare stress results between the models using the F-M and F-F bearing conditions. Almost no difference is observed between these results.

The stresses on both the cross beam web and the bracket flange are high only when the load is on the nearby lane (In-1), and are almost zero when the load is on the far lane (Out-1). This indicates that the stresses at the cross beam web and the bracket flange are induced solely by bearing force acting on the bracket. Both figures show that stress distributions are very high at the center of the bracket; this reflects the influence of the diaphragm inside the box cross beam.

The fillet weld between bracket U-flg. and cross beam web can be considered to be a cruciform joint (see cross section in **Fig. 6(b)**). The joint is transverse non-load-carrying type when subjected to a load in the bracket flange. In this case, a possible failure mode is crack initiated at the weld toe on the bracket flange. Meanwhile, the joint is a transverse load-carrying type when subjected to a load in the cross beam web. In this case, cracks could be expected to develop either at the weld toe on the web plate or at the weld root. The stresses in the cross beam web and the bracket flange, shown in **Fig. 21**, indicate that the joint is subjected to forces in both members. Therefore, all three failure modes discussed above possibly to develop at the joint. However, it was found that the majority of cracks developed from weld root.
The possible reason for crack developing at weld root could be the large root gap resulted from bad plate alignment. In order to explain this, refer to Fig. 6(b), the bracket flange and the cross beam web were designed to be connected by fillet weld with leg length of 6 mm. By the width of the slot hole (25 mm) and the thickness of bracket flange (22 mm), if the flange is properly fitted in at the center of the slot hole, then the weld should have root gap of 1.5 mm. However, there is high possibility that the flange was aligned at the bottom side of the slot hole. Then the weld at the upper side of the flange plate would have root gap as large as 3.0 mm. In addition to the large root gap, the actual leg length is reduced to 3 mm that is only half of 6 mm of designed leg length as shown in Fig. 22. The short of weld leg length is followed by the decreasing of strength of the weld connection and the large root gap causes the development of crack at weld root.

According to stress results and consideration of plate alignment, it can be deduced that the cracks at bracket U-flg. are caused by the high bearing forces acting on the brackets. The cracks mostly developed at weld root could be due to the large root gap resulted from bad plate alignment.

4) Stresses and cracks at beam-to-column connections

Fig. 23 shows stress distributions on the lower flange of beam to column connection at pier P2-2 (produced by the axle load on each lane at section 9, directly above pier P2). These stresses are in the direction perpendicular to the weld bead and were taken from the first nodes next to the connection, which are 12 mm apart from the connection (indicated by the reference line). The figure also shows a comparison between results for the F-M and F-F conditions and it demonstrates the same stress distributions between these two cases.

It can be seen that the stresses are higher away from the center of the flange, and highest near the webs. This behavior is caused by the non-uniformity of shear deformations across the width of the flange, and it is called “shear lag”. The effect of shear lag is very large in case of loading on lane Out-1. In addition to the effect of shear lag, the stresses at the connection are also dependent on load location in the transverse direction. As can be seen in the figure, the stresses are reversed from compression to tension when the load moves from lane Out-1 to In-1. This also increases the stress range. These results lead to the conclusion that the fatigue cracks in beam-to-column connection are most likely to occur near the webs and directly under the webs.

As discussed previously, the connection type 1 is prone to contain large weld roots, shown in Fig. 8 as an example. In addition to the shear lag problem, these large weld roots form stress concentration, making the weld connection very sensitive to fatigue crack propagations. Based on these results, both Crack 1 and Crack 2 in beam-to-column connection type 1 are attributable to the effect of shear lag and the large weld roots inside the weld connection itself.

6. CONCLUSIONS

The finite element analysis of the bridge 3D model and field tests were conducted to identify the causes of fatigue cracks in the object bridge. The results led to the following conclusions.

- Bearing force behaviors: The fixed-fixed bearing condition due to the corrosion damage induces large horizontal bearing forces which do not exist in original fixed-movable bearing condition. This is followed by the increasing of stress ranges and number of stress cycles at notched details of cut-off girders. The stress configurations near cracks
at brackets and beam-to-column connections are not affected by the change of bearing condition.

The bearing force distributions at each girder were also found to be affected by the existence of steel frame piers. The bridge model should incorporate the steel piers in order to accurately calculate the bearing forces in the system.

- **Cause of cracks at cut-off girder ends:** The fixed-fixed bearing condition resulting from the corrosion damage is one of the dominant causes of cracking in addition to the out of plane deformation in the web plate. Under the fixed-fixed bearing condition, the notched details are always subjected to higher range and frequency of stress cycles compared to the originally designed fixed-moved bearing condition.

- **Cause of cracks at brackets:** For cracks at slot corners, the weld toe at slot corners are sensitive to fatigue cracking because of the high stress ranges induced by the bearing forces acting on the brackets and the deformation of the cross beams. Moreover, the weld toes are always subjected to significant stress regardless of position of vehicles on the roadway since the cross beams deform under all loading positions.

For cracks at bracket U-flg., cracks are caused by the high bearing forces acting on the bracket. The possible reason for that cracks mostly developed at weld root is the large weld root resulted from the poor plate fit up.

- **Cause of cracks at beam-to-column connections:** Fatigue cracks found at beam-to-column connection are attributable to the effect of shear lag and large weld root. The effect of shear lag is very large making the weld connection near cross beam web subjected to high stresses. Furthermore, the structural detail of type 1 connection is prone to have large weld root inside the weld connection.

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**REFERENCES**


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鋼製橋脚を有するプレートガーダー橋の疲労損傷原因同定

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近年、都市内道路橋およびそれらを支える橋脚に多くの疲労損傷が報告されている。これらの道路構造物は、厳しい立地面条件のため、複雑で特有の構造・線形を有している。このため、道路構造物の応力・変形挙動は複雑となり各部に発生した疲労損傷の原因が明確になっていない。そこで本研究では、首都高速道路の代表的なプレートガーダー橋を対象として、応力・変位の実験測定、および橋脚を含めた詳細な立体 FEM 解析によって対象構架の応力・変形挙動を明らかにし、疲労損傷の発生要因を同定した。