

Failure Analysis of Kutai-Kartanegara Bridge's Hanger Clamps from Fracture Mechanics Viewpoint*

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On November 26, 2011 Kutai-Kartanegara Suspension Bridge in Indonesia collapsed. The collapse was triggered by failure of the clamp of cable band that connects hanger with main cable in the middle of the center-span. The national investigation team report cited the failure as a result of stress accumulation on the clamps that have been weakened by fatigue, initial fracture and corrosion. In this paper, we investigate more detail the possibility of shear brittle fracture of the clamp's pin from a viewpoint of linear fracture mechanics by utilizing the measured Charpy absorbed energy. Several possible scenarios of defect sizes and combined stress conditions were assumed. The analysis shows that the shear brittle fracture could occur even under low shear stress level when several unfavorable conditions occur simultaneously.

Key Words : cable band, clamp pins, shear failure, linear fracture mechanics, brittle fracture

***) Japanese version of this paper was published on JSCE Structure Engineering A1 Vol.69, No.2, 410-415, 2013**

1. INTRODUCTION

The report of national investigation team on the collapse of the Kutai-Kartanegara Bridge, the longest suspension bridge in Indonesia, released in January 2012, reveals that accumulation of design faults, lack of maintenance and improper retrofit work have sequentially cause the bridge collapse. In particular, the report emphasizes on the stress accumulation of the hanger clamps as the triggering mechanism of the collapse¹⁾. Post-accident investigation of the clamps has shown that they might have been weakened by corrosion or cracking and fatigue. The report also questions selection of high strength ductile iron FCD-600 that has low energy impact absorption and poor toughness as material of the clamps.

Based on the bridge design document, accident investigation report and field survey conducted by the first author immediately after the accident, the authors share the view of the national investigation team that performance of clamp should be considered as possible cause of the collapse. However, in our opinion, such conclusion should be assisted by

fracture surface analysis, and such analysis, to the best of authors' knowledge, has not been performed. This is partly because the main part of the hanger clamp from the middle of center span that triggered bridge collapse fall into the river, while the other parts are still attached to the suspension cable and remained inaccessible for analysis.

In this note, we investigate the possibility of brittle fracture of the clamp from linear fracture mechanics viewpoint. Fracture toughness of the clamp is estimated using results from the Charpy impact values obtained from material test of the remaining clamps. The possibility of brittle shear fracture is examined by assuming initial defects at the base of pins and by considering fracture toughness of the material under influence of combined tensile and shear stress on the clamp.

2. BRIDGE DESCRIPTION AND COLLAPSE INCIDENT

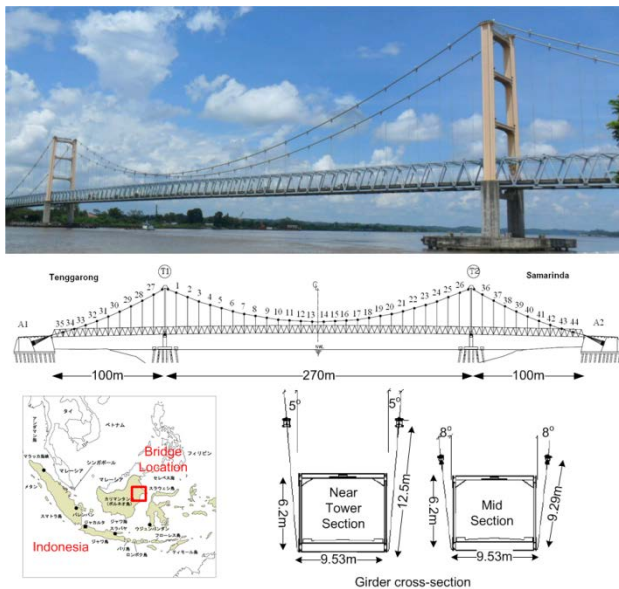


Fig.1 Kutai-Kartanegara Suspension Bridge

Kutai-Kartanegara Bridge was a suspension bridge located in the city of Tenggarong in Kutai-Kartanegara Regency, East Kalimantan Province, Borneo Island, Indonesia. The bridge crossed over Mahakam River and connected Tenggarong city and Tenggarong Sebrang, a city next to Samarinda, capital of the province.

The total length of suspension part was 470m consisting of 100m side-spans and 270m center-span. Pylons were 37m tall made of steel and mounted on 15m high concrete column to give the main cable's sag ratio of 0.21. Stiffening girder was made of *Warren-through* steel truss without vertical and was continuous at the pylon supports. The suspension cables consisted of 19 galvanized spiral wire strands with diameter 58mm each and were parallelly bundled without twisting following the *Equal Lay* method. The cables continued from the center span over the pylons saddle into the suspended side span and splayed over the cable bents located on top of abutments, which besides supporting the main cables, also supporting the side span stiffening truss. At both ends of side span, the main cables were anchored to the anchorage blocks, which were supported by battered pile foundations (Figure 1).

The truss girder was connected to main cables by parallel hangers made of steel rod with 63mm in diameter. In total, there were 44 hangers on one side and they were spaced at every 10m. Hangers and stiffening girder were connected by rocker bearing at the lower chord of the truss (Figure 2). Meanwhile, clamps are used to connect the main cables and hangers.

The main cable's clamps consisted of cable bands, horizontal bar, two sides clamp arms and rocker bearing. Since the horizontal distance between the two main cables, 12.1m, is larger than the width of the stiffening truss girder, 9.53m; the hangers have vertical inclination angle. The angles were small for hangers near pylons and become larger for hangers in the middle of

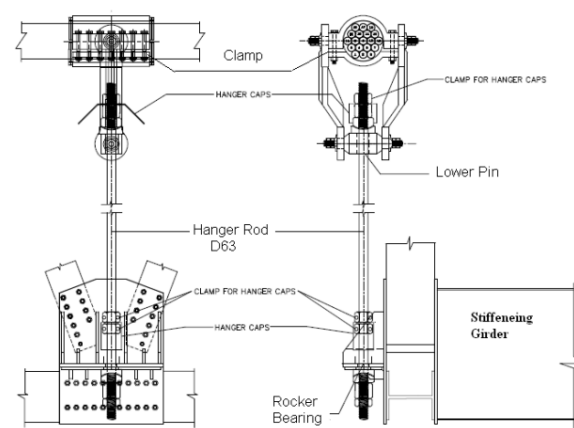


Fig.2. Schematic figure of hanger connection to main cable and stiffening

the center-span.

After construction completion in 2001, the bridge was opened to traffic and high traffic volume was recorded since the bridge was practically the main connection between the two cities. In the beginning of year 2006, there was a report to the local authority about gradual movement of anchorage blocks towards the main span²⁾. The report mentioned that the movement had created a gap in the expansion joint between abutments and side-spans. The reported anchorage blocks movement allegedly influenced the main-cables on the side-span and the pylon top displacement, and as a result top of the pylon inclined towards the center-span. This inclination resulted in the sagging or negative camber of the center-span. In addition, some clamps that connect main-cables and hangers were reported to have moved from their original position as a direct consequence of main-cable increasing sag.

Following the report, a recommendation to improve bridge condition was issued. The recommendation included the following actions: (1) adding sandbags as counterweights on the anchorage blocks to prevent further movement of the anchorages, (2) replacing the existing expansion joints between side-spans and bridge abutments, (3) restoring camber of girder center-span and (4) tightening bolts and connections on hangers and girder after restoring the camber.

On October 12, 2011 the contract was awarded to a contractor to undertake recommendation number 3 and 4. The bridge collapsed on November 26, 2011, when maintenance work aimed at restoring the camber to its initial position by lifting the girder and tightening the hanger-to-girder connections was performed. The maintenance work allegedly led to a failure to one of the hanger-to-suspension connections in the middle of the center span (i.e. hanger 14 in Figure 1). The failure was quickly followed by the other hanger-to-suspension connections and led to the progressive collapse of the center-span. In the end only the pylons and titled side-spans remained. Twenty three people confirmed dead while about 17 people were proclaimed missing in the accident, since the bridge was opened to traffic

during the maintenance work.

3. STRUCTURAL CHARACTERISTICS AND DESIGN STRENGTH OF THE CLAMPS

Since the collapsed was initiated by the clamp failure, the present study focuses on structural performance of the clamp by investigating the load transfer mechanism, material and strength of the clamp. The clamp consists of the following structural elements:

- 1) Cable band, two-half cylinder shape cast-iron with inner and outer radii of 15.1cm and 18.85cm, respectively. The cable-band wraps the main cable with the length of 56cm. The upper and lower halves are connected by bolts on both sides of the cable band.
- 2) Upper pin, a rod with two diameters and conical connection. The rod with larger diameter and the conical connection are made on cast iron and were cast together with the cable band. Meanwhile the rod with smaller diameter is made of threaded steel bar and driven into the conical connection part. The threaded steel rod also connects the cable band to the clamp's side arm and is locked by a nut at the end.
- 3) Side arm, the main body of clamp that connects the cable band and hanging bar. Each clamp has two side arms made of cast iron with the length of 75cm.
- 4) Hanging bar, a horizontal rod that connects the side arm to the hanger. Hanging bar has three shapes: rectangular shape in the middle, rod on both ends and conical shape in between. The rod end is called lower pin and made of cast iron.
- 5) Rocker bearing. The hanger goes through a hole made inside the hanging bar and locked at the

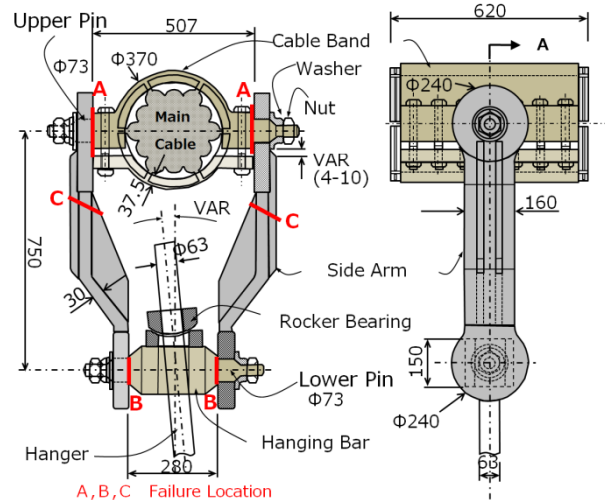


Fig.3 Schematic figure on failure locations on the clamp

end by nut and contra nut. A rocker bearing is placed below the contra nut to facilitate incline movement of the hanger. The rocker bearing, nut and contra nut are all made of cast iron.

The load transfer mechanism on the clamps can be described as follows. Firstly, the girder dead load and traffic live load are transferred through the hanger rod to the hanging bar by rocker bearing. The hanging bar transfers the load to both side arms through the lower pin and the conical shape. This loading transfer mechanism implies that the lower pin and the conical shape are under extreme shear stress for most of the hangers with small vertical inclination angle. For hangers with larger vertical inclination angle such as hangers in the middle of the center-span, however, the pin experiences combined stress from shear and moment. Secondly, the side arms transfer the load from hanging bar to the upper pin. Finally the upper pin transfers the load from side arms to cable band and main cable. Similar to lower pin, the upper pin and the conical shape are under extreme shear stress during load transfer mechanism. With such structural characteristics, one can recognize that the lower and upper pins are the most critical structural elements in loading transfer mechanisms.

Figure 4 shows photographs of the failed clamps located in the middle of the span (i.e. clamps number 14). The photographs show shear failure occurring on the upper-pin and on the lower-pin. The clamps were made of ductile cast Iron FCD 600-3. Pin of the hanger clamp was cast together with cable band as shown in Fig.3. The pins, placed on both sides of the main cable's band, were designed to resist shear force caused by axial load on the hanger, and the shear strength is calculated in accordance with AASHTO LRFD¹⁾ as:

Yield condition:

$$R_{p,y} = \phi \cdot A_p \cdot f_{y,p} \quad (1)$$

Ultimate fracture condition:

$$R_{p,u} = \phi \cdot A_p \cdot f_{u,p} \quad (2)$$

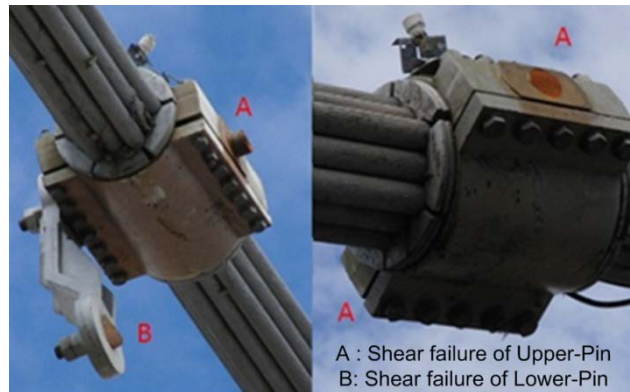


Fig.4 Photos of damaged clamps

Here, A_p denotes cross-sectional area of the pin, ϕ is the strength reduction factor; while $f_{y,p}$ and $f_{u,p}$ denote the yield and ultimate tensile stress of the pin, respectively. Diameter of the pin is $D=73\text{mm}$. In design, the $f_{y,p}$ and $f_{u,p}$ were 370MPa and 600MPa, respectively. However, from the laboratory test of the specimen, the actual values of $f_{y,p}$ and $f_{u,p}$ were found to be 595MPa and 781MPa, respectively.

Since the target of analysis is the shear strength, the strength reduction factor of $\phi = \tau_y / f_f = 1/\sqrt{3} = 0.58$ is used. As a result, the design shear yield resistance $R_{p,y}$ and ultimate shear resistance $R_{p,u}$ become 898kN (91.65tf) and 1,456kN (148.54tf), respectively. Meanwhile, the actual yield and ultimate shear resistance of the pin were 1,444 kN (147tf) and 1,896 kN (193tf), respectively. Table 1, shows the measured values obtained from the laboratory test of a damaged clamp specimen and comparisons with mechanical properties of the standard material (FCD 600-3).

In the retrofit work plan³⁾, the maximum jacking load on the hanger was limited to 595kN (60.72tonf) and the shear stress on the pin was considered to be 150Mpa. Had the retrofit work been carried out according to the plan, the shear forces acting on the pin during the jacking would have not exceeded about one-third of the actual ultimate strength (based on the tensile strength of the specimen test). Considering this fact, even by taking into account the coefficient of non-uniform shear force of 2.0 and the reduction of cross section due to possible corrosion and fatigue cracks, it was unlikely that ductile shear failure had taken place.

Therefore, we investigate the possibility of brittle shear failure, by assuming the existence of an initial crack-like defect as will be described in the following sections.

4.ESTIMATION OF FRACTURE TOUGHNESS OF CLAMP MATERIAL(FCD 600-3)

As explained in Table 1. JIS G 5502 does not specify the Charpy impact value requirement for material FCD 600-3. The result of material testing of the damaged clamp provides the Charpy impact value (v_E) of 4.27J. In reference⁴⁾, it has been reported that FCD400 material tested in room temperature (20°C) could have the Charpy impact value of 15-20J. In the specification of material FCD-400 there is a provision for minimum impact value of 14J. Judging from the test results, one can clearly see that the FCD-600 used as the clamp material indeed has low material toughness. The Charpy impact value can be used to determine ductility of a material; however,

Table.1. Mechanical properties of the material clamp

		JIS G5502	Test results from damaged specimen	Reference FCD400-18
Tensile Strength (MPa)		600 or more	781	400 or more
0.2%Proof Stress (MPa)		370 or more	595	250 or more
Elongation (%)		3 or more	2.2	18 or more
Charpy Energy Absorbed v_E	Test Temperature (°C)	Not Specified	25	23±5
	Average of 3 (J)		4.27	14
	Individual Value (J)		—	11 or more
Brinell Hardness:HB		170~270	261	130~180
Composing Materials		Perlite and Ferrite	—	Ferrite

basically it does not have theoretical relation with fracture mechanics. From fracture mechanics point of view we must use fracture toughness performance index such as J integral (J_c) and linear fracture toughness (K_{IC}) to describe the possibility of brittle fracture.

In this investigation, the only available information on toughness of the material is the Charpy impact value obtained from the laboratory test of the damaged clamp. Therefore, using this value an attempt to estimate fracture toughness K_{IC} is sought in order to examine the condition for brittle fracture of the clamp by linear fracture mechanics approach.

There are three types of crack propagation under different loading condition (Fig.5), namely, opening (Mode I), in-plane shear (Mode II) and out-of-plane shear (Mode III). Shear failure of the pin of the clamp can be categorized as mode II.Among the three

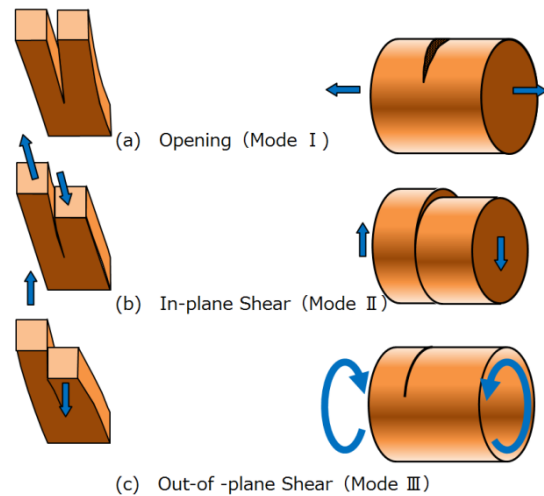


Fig.5 Loading modes that induce crack

modes, empirical formulations to estimate K_{IC} using the Charpy impact value for opening mode (i.e. K_{IC} for mode I) have been proposed in numerous studies. However, not so many studies have been proposed the formulation for in-plane shear (mode II).

Several studies have proposed empirical formulations to estimate K_{IC} from the Charpy impact value depending upon the chemical composition of the steel and the steel yield strength. Bannister⁵⁾ proposed a direct relationship between Charpy impact value $\sqrt{E}$ and linear fracture toughness K_{IC} as:

$$K_{IC} = 19\sqrt{\sqrt{E}} \quad (K_{IC} : MPa\sqrt{m}, \sqrt{E} : J) \quad (3)$$

Barsom and Rolfe⁶⁾ proposed the following empirical relationship for steel material with yield strength between 760 and 1700 MPa:

$$\left(\frac{K_{IC}}{\sigma_y}\right)^2 = 0.64 \left(\frac{\sqrt{E}}{\sigma_y} - 0.01\right) \quad (4)$$

$$(K_{IC} : MPa\sqrt{m}, \sqrt{E} : J, \sigma_y : MPa)$$

Roberts and Newton⁷⁾ proposed the following relation:

$$K_{IC} = 0.54\sqrt{E} + 55 \quad (K_{IC} : MPa\sqrt{m}, \sqrt{E} : J) \quad (5)$$

In the INSTA Technical Report⁸⁾, the relationship between fracture toughness and the Charpy impact value for a reference plate thickness of $t = 25\text{mm}$ is described as:

$$K_{mat,25} = 12\sqrt{\sqrt{E}} \quad (K_{IC} : MPa\sqrt{m}, \sqrt{E} : J) \quad (6a)$$

while for any t thickness of the plate, relationship becomes:

$$K_{mat,t} = \left[(K_{mat,25} - 20)(25/t)^{1/4} \right] + 20 \quad (6b)$$

For rolled steel plate carbon with the yield stress of about 235-323MPa, Yajima et al⁹⁾ proposed the relationship between fracture toughness and Charpy impact value as:

$$\frac{K_{IC,t=50mm}^2}{\sigma_{YO}^2} = 40\sqrt{\sqrt{E}} \quad (7)$$

where σ_{YO} is the specified yield point of the base material in kgf/mm^2 . $K_{IC,t=50mm}$ is the fracture toughness at $t(\text{mm})$ thickness plate, and $\sqrt{E}$ is the Charpy impact value. Note that $1\text{kgf} \cdot \text{mm}^{-3/2} = 0.310114\text{MPa}\sqrt{m}$ and $1\text{kgf} \cdot \text{m} = 9.8\text{J}$.

Expressions and relationships used in the above-mentioned studies are typically for material with large value of Charpy impact value at the room temperature. Therefore, question remains on the applicability of the formula on material with low fracture toughness (about 4) as measured from test of the clamp.

Nevertheless, Table 2 shows the values of fracture toughness for Mode I estimated by the above formula. And despite some variations, the estimates are generally within the range of $40 - 60\text{MPa}\sqrt{m}$.

Table.2 Estimates of fracture toughness by empirical formula

Fracture toughness estimate	Eq.(3)	Eq.(4)	Eq.(5)	Eq.(6)	Eq.(7)
K_{IC} ($\text{MPa}\sqrt{m}$)	39.3	Not applicable ^{*)}	57.3	51.2	57.4

*) $\sqrt{E}/\sigma_y = 4.27/595 = 0.0072 < 0.01$

Therefore, in the subsequent analysis we assume the lowest $K_{IC} = 40\text{MPa}\sqrt{m}$ as the representative value.

It should be noted, however, that the fracture toughness values are only for load shape opening mode I, while the shear failure of the pin is in fact an in-plane shear opening (mode II). Therefore, we need to consider the relationship between fracture toughness of in-plane shear opening (mode II) and that of the load shape opening mode I.

The relationship between K_{IIC} and K_{IC} have been investigated by many researchers and the results vary depending on the test temperature and steel grade. Erdogan and Sih¹⁰⁾ reported the value of $K_{IIC} / K_{IC} = 0.71$ obtained by the maximum tangential stress theory as one of the criterion of brittle fracture. From investigation on 0.04% carbon steel, Yokobori et al.¹¹⁾ suggested $K_{IIC} / K_{IC} = 0.7-0.9$. Shih¹²⁾ suggested the value of $K_{IIC} / K_{IC} = 1.09$. From the test on HT50 at room temperature, Takamatsu and Ichikawa¹³⁾ suggest the value of $K_{IIC} / K_{IC} = 0.95$. Therefore, in the following, we compare fracture toughness of mode I and mode II under $K_{IIC} / K_{IC} = 0.7$ and $K_{IIC} / K_{IC} = 1$ as two extreme conditions.

5. ANALYSIS ON THE POSSIBILITY OF BRITTLE FRACTURE ON THE PIN

The precise loading conditions of the hanger rods during retrofit work remains unknown, since the exact amount of force that had been transferred to during the jacking process remains unknown. Moreover, fracture surface analysis of the clamp that triggered the collapse could not be carried out, since parts of the cable band and pin clamp on the center of the span are still inaccessible, while the hanging bar and side arms had fallen into the river.

In the following, we shall discuss the cause of shear failure at the base of the pin by assuming 595.06kN as the loading condition that is the maximum limited load of jacking work as indicated in the retrofit work plan. Analysis is conducted by assuming the worst load scenario that is eccentric load only on one side of the side arms, and that shear strength of the pin is as described in the Statement Summary of design calculation²⁾.

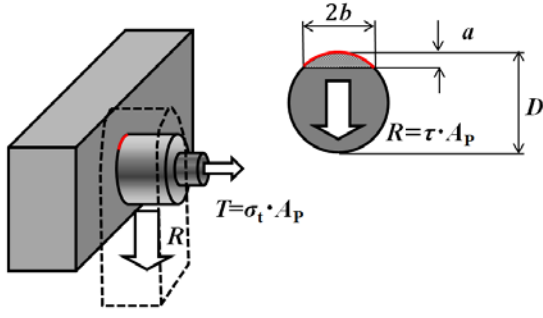


Fig.6 Loading mechanism on the pin

It should be noted that ductile shear fracture occurs when the shear force applied on the net sectional area of the pin (i.e. effective sectional area after considering corrosion loss or fatigue crack propagation) is equal to the hanger load. On the other hand, brittle shear fracture occurs when the stress intensity factor on the tip of crack-like defect of their-plane shear crack (mode II), denoted as K_{II} , is equal to or greater than the pin material fracture toughness (K_{IIC}).

To calculate the K_{II} , FEM analysis is normally employed¹⁴. However, since we focus on examining schematically the possibility of shear brittle fracture of the pins, an approximate expression for the stress intensity factor K_{II} for a bar having a surface crack is considered reasonable amid the complexity of FEM analysis¹⁵. For approximation, the stress intensity factor on a cracked rectangular cross-section, instead of circular section given as the following formula¹⁶, is applied.

$$K_{II}(\alpha) = F_{II}(\alpha) \cdot \tau \cdot \sqrt{\pi a} \quad (8)$$

where

$$F_{II}(\alpha) = 2.13 - 11.03\alpha + 35.01\alpha^2 - 59.44\alpha^3 + 52.09\alpha^4 - 17.73\alpha^5 \quad (9)$$

In expression above, $\tau = P/A$, $\alpha = a/D$, P denotes the axial load in hanger rod, A is the cross-sectional area of the pin, a is the crack depth and D is the height of the pin.

Cross-sectional area of the pin base is designed only for shear forces due to vertical load acting on the hanger rod. However, as shown in Fig.3 both ends of the pin are clamped by washer and nut to prevent it from getting out of the side arms. When horizontal force is applied on the hanger rod, as shown in Fig.6, the acting force will induce not only shear but also normal stress. This means stress evaluation under combined forces is required. In such a case, tensile stress σ ; stress intensity factor of the round bar with surface crack is given by the following equation¹⁷.

$$K_I(\alpha) = F_I(\alpha, \beta) \cdot \sigma \cdot \sqrt{\pi a} \quad (10)$$

where

$$F_I(\alpha, \beta) = (1.122 - 0.230\beta - 0.901\beta^2 + 0.949\beta^3 - 0.280\beta^4) \times (1.0 + 0.314\alpha - 2.536\alpha^2 + 36.72\alpha^3 - 106.048\alpha^4)$$

(11)

Note that $\alpha = a/D$ and $\beta = a/b$, where a denotes the crack depth, $2b$ denotes surface crack width. However, the formula applies only to $\alpha \leq 0.25$ and $\beta \leq 1.0$.

Critical condition for the occurrence of brittle fracture under such combination of loading mode is generally represented by the effective stress intensity factor K_{eff} as follows:

$$K_{eff} = \sqrt{K_I^2 + K_{II}^2 + \frac{1}{1-\nu} K_{III}^2} \leq K_C \quad (12)$$

Since the possible rotation of the side arms and the pin is small, the out-of-plane shear load due to torsional loading in above equation can be ignored. Hence, $K_{III} = 0$ and the condition for brittle fracture for mode II under combined axial and shear forces becomes:

$$K_{II} = \sqrt{K_C^2 - K_I^2} \quad (13)$$

From the above equation, the crack depth a and the critical shear stress τ_{cr} for the occurrence of brittle fracture is given as:

$$\tau_{cr} = \frac{\sqrt{K_C^2 - K_I^2}}{F_{II} \cdot \sqrt{\pi a}} \quad (14)$$

Fig.7 and Fig.8 show the calculated relationship between the brittle shear strength of the pin R_{uc} and crack depth a for the varying normal tensile stress σ_t of the pin for $K_{IIC}/K_{IC} = 1.0$ and $K_{IIC}/K_{IC} = 0.7$, respectively. The figures also plot the relationships between ductile shear strength and crack depth. Note that the applied normal tensile stress was not considered in the original design of the pins and they are only assumed stress conditions. In these figures, the ultimate shear strength R_u , $R_{u,real}$ was calculated using Eq.(1) and the yield shear strength R_y , $R_{y,real}$ was calculated using Eq.(2). Here, R_u and R_y are calculated for the nominal specified shear strength and $R_{u,real}$ and $R_{y,real}$ are for actual ones, i.e. obtained from material test of damaged clamp specimen, respectively. These shear strengths are ductile failure

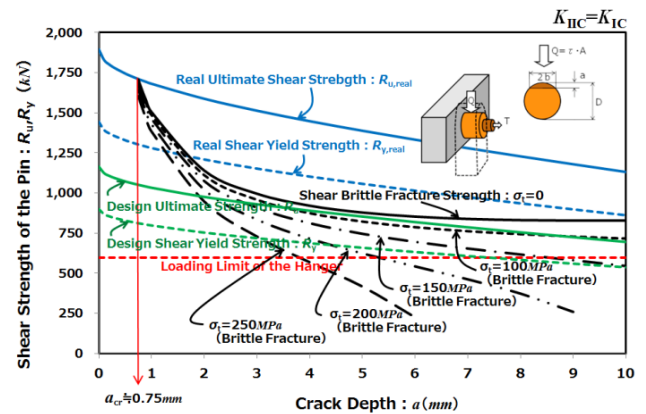


Fig.7 Shear strength of the Pin under pure shear and combined shear-tensile stress for $K_{II}=K_I$

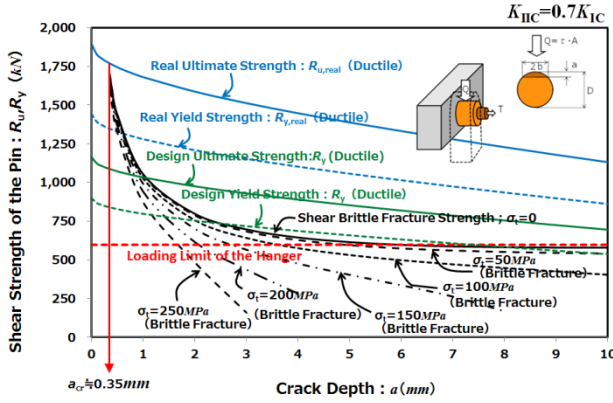


Fig.8 Shear strength of the Pin under pure shear and combined shear-tensile stress for $K_{IIc}=0.7K_{IC}$

strength calculated by considering the reduction in effective cross sectional area due to crack propagation of the upper part of the pin as shown in Fig.6.

Concerning brittle shear strength, effects of combined stress on the pin shear strength are illustrated by varying several values of normal tensile stress of the pin (σ_t). The figure shows that brittle fracture occurs if depth of the crack a is more than the critical depth (a_{cr}). This alters the fracture mode from ductile manner to brittle manner. From this point onward, the apparent shear strength decreases rapidly with small crack depth increase.

Furthermore, when normal tensile force and the shear forces are both applied, the rate of reduction in apparent shear strength with respect to the crack depth becomes larger, and the shear strength decreases more rapidly with an increase in the normal tensile force on the pin.

If the normal tensile stress does not take place (as is assumed in the design), Fig.7 shows that brittle fracture will occur when the crack depth reaches the critical crack depth of at $a_{cr}=0.75\text{mm}$ under the condition of $K_{IIc}/K_{IC}=1$. Note that the rate of shear strength reduction is increasing when the assumed normal tensile stress is larger than 150MPa. This suggests that a defect with the depth of few millimeters can rapidly initiate the brittle fracture of the pin when the assumed normal tensile stress is higher than 150MPa. It is important to point out that the normal tensile stress higher than 150MPa is unlikely to occur in the loading condition so that the probability of brittle fracture occurrence in this scenario is quite small.

On the other hand, in the case of $K_{IIc}/K_{IC} = 0.7$ (Fig 8), the estimated values of fracture toughness are lower than that of $K_{IIc}/K_{IC}=1$. The critical crack depth reduces to $a_{cr}=0.35\text{mm}$ and the shear strengths of the pin drop rapidly even when the assumed normal tensile stress is slightly larger than 50MPa. This suggests that when the jacking load on the hanger is

the maximum limited load of 595.06kN, a small defect with crack depth of slightly over 0.5mm can rapidly initiate the brittle fracture under normal tensile stress slightly higher than 50MPa. Such normal tensile stress condition is likely to occur in the loading condition, even under controlled loading condition.

Accordingly, for both cases (Fig. 7 and 8), the brittle failure could occur when the maximum jacking load creates stress combination larger than the assumed combination of tensile and shear stresses on the pin. For the case in Fig.8, even a small tensile force can cause larger combined stresses while in the case of Fig.7 larger tensile force is required. Therefore, the possibility of occurrence of brittle fracture is larger for the case in Fig 8 than it is in the case of Fig.7.

It should be mentioned that during jacking process the truss girder was lifted on one side. This process would have changed the inclination angle of the hanger due to geometric adjustment. As a result, there was a possibility that jacking load acted only on one side of the pin and induced secondary lateral moment. Such secondary moment due to the lateral force of the rod also may evoke secondary tensile stresses. This would have further increased the normal tensile stress and created an unfavorable stress condition on the pin.

Note that the results above are valid only when the following preconditions and assumptions are satisfied: some sort of crack-like defects at the base of the pin had been propagating due to repeated loading and they were not detected due to lack of maintenance, normal tensile stress was not considered when designing the pin and the fracture toughness at room temperature of the pin was significantly low. These preconditions and assumptions are in line with the national investigation team report, which concluded that accumulation and combination of inadequate design, lack of maintenance and improper retrofit work have led to the bridge collapse.

6. CONCLUDING REMARKS

The present study discusses the possibility of shear brittle fracture of the pin under the loading condition in the repair work by means of linear fracture mechanics. Utilizing the information from Charpy impact absorbed energy of the failed clamps; the fracture toughness of the high strength ductile iron FCD-600 used for the pins and clamps is estimated. The stress condition on pins and clamps were analyzed from fracture mechanics viewpoint by comparing the fracture toughness and the estimated actual stress on the structure. The result shows that even though the load on the hanger is lower than the

maximum jacking load, a brittle fracture may occur under the combined normal tensile and shear stress, when a crack-like defect exists on the base of the pin.

The collapse of Kutai-Kartanegara Bridge can be classified as progressive type of collapse. More specifically, it can be considered as zipper type of collapse, where failure of one structural component leads to rapid failure of other structural components and result in structural collapse. Progressive collapse usually has two distinct characteristics, namely, having triggering mechanism and having collapse promoting features¹⁸⁾. The former is the main cause of failure of one structural component while the latter is the main cause of failure propagation to the whole structure. Both characteristics are equally important in that the absence of one would not lead to the structural collapse.

In the case of Kutai-Kartanegara Bridge the triggering mechanism of the collapse is the jacking work that may have caused overstress on the connections, while the collapse promoting features include poor connection conditions due to imperfect design, questionable material selection and poor maintenance.

ACKNOWLEDGEMENT AND DISCLAIMER

We gratefully acknowledge the support from Indonesian institutions: Institut Teknologi Bandung, PT.BTU and local government of Kutai-Kartanegara Regency for facilitating the post-incident survey and for sharing the data on bridge design and retrofit work. This study was partially supported by cSUR GCOE Univ.of Tokyo to the first author under Grant ID (GSRR11005). Opinions, findings and conclusions in this study are those of the authors and do not represent those of institutions mentioned above. This study is solely for academic purpose and accordingly should be exempted from any legal process and consequences that may incur following the incident.

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