FATIGUE ANALYSIS OF K-JOINT IN A HALF-THROUGH CONCRETE-FILLED STEEL TUBULAR TRUSSED ARCH BRIDGE IN CHINA

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SUMMARY

Compared with hollow section structure, concrete-filled steel tubular (CFST) structure is expected to improve stress conditions, delay fatigue damage, and enhance fatigue life. It has become an important bridge type in China since 1990. Until now about 450 CFST arch bridges have been built. In April 2013, cracks in the connections of arch chord and brace of a half-through CFST trussed arch bridge with the main span length of 136 m were found during the daily inspection, which seriously damages the safety of the bridge. In this study, numerical analyses to obtain stress concentration factors (SCF) and hot spot stress were conducted, and the number of cycles to failure was estimated for the K-joint with the longest crack.

Keywords: *Concrete-filled steel tubular, trussed, arch bridges, fatigue life, SCF.*

1. INTRODUCTION

CFST arch bridges have become popular in China. Until now about 450 CFST arch bridges have been built. Bosideng Yangtze River Bridge with the main span length of 530 m sets a new world record for the CFST arch span, which was completed in 2013. CFST can be used to improve stress conditions, delay fatigue damage, and enhance fatigue life. But, the intersecting line with full penetration butt welds in CFST joint is the weak part in the whole structure, due to the axial stiffness of brace is much bigger than the radial stiffness of chord tube, which leads to stress concentration and weld defects [1, 2]. Furthermore, the self-equilibrated residual stress in vertical and lateral directions induced during welding process of CFST joint usually reaches the yield point of the material, which aggravates the fatigue issue [3]. Shao [4] presented general remarks of the effect of the geometrical parameters on the stress distribution in the hot spot stress region for tubular T-joints and K-joints under brace axial loading. It was found that chord thickness has remarkable effect on the stress distribution for both T-joints and K-joints, while brace thickness has no effect on such stress distribution, and the diameter ratio between the brace and the chord has different effects on the stress distribution for tubular T-joints and K-joints. Sakai [5] described an experimental study on the ultimate strength and fatigue strength of concrete-filled and additionally reinforced tubular K-joint of truss girder. The static test results indicated that the ultimate load of the concrete-filled specimen was about twice as large as that of the concrete non-filled specimen. However, in order to reduce localized stress at the weld toe of the joint, reinforcement was needed. The fatigue test results indicated that the specimen whose centerlines of members meet at one point and the specimen with gusset have enough fatigue strength for the joint of the chord and the diagonal members. Based on fracture and damage mechanics, Haldimann [6] presented a methodology for the determination of allowable initial sizes of casting defects as a function of the required fatigue resistance of the welds. The relative influence of the main parameters was quantitatively discussed, and recommendations for design were given. Concerning welded circular hollow section (CHS) K-joints and concrete-filled K-joints under axial loading, Tong [7] experimentally revealed the SCFs at the concrete-filled K-joints tended to be more uniform and significantly smaller compared with CHS K-joints. Chen [8] experimentally investigated the SCFs of welded CHS T-joints and concrete-filled T-joints under axial and in-plane bending loading, and demonstrated that the stress concentration factors (SCFs) for T-joints at each loading conditions tended to be conservative compared with present design codes. Wang [9] showed concrete-filled T-joints have a much lower stress concentration factor and consequently have better fatigue strength than the CHS T-joints. Xu [10] studied the SCFs along the intersection of chord and brace of thin-walled concrete-filled T-joints, Y-joints, K-joints, and KT-joints under axial tension loading. The results indicated that the stress distribution is mainly determined by joint type, while the chord thickness has little effect on it. However, results of these studies have not been reflected in specifications yet.

Since the history of CFST arch bridge construction is short, at the present time, experimental verification of its fatigue performance is not sufficient. Consequently, fatigue checking calculation of CFST joints by S-N curve and provisions are not given by the current specifications in China. Some structural types to prevent fatigue are provided and the maximum nominal stresses are recommended by the specifications. In April 2013, cracks seriously damaging the structural safety in the connections of arch chord and brace of a half-through CFST trussed arch bridge were found during the daily inspection. In this study, numerical analyses to obtain stress concentration factors (SCF) and hot spot stress were conducted, and the number of cycles to failure was estimated for the K-joint with the longest crack.

2. OUTLINE OF BRIDGE AND CRACKS

2.1. Bridge

The bridge studied is a half-through CFST trussed arch bridge shown in Fig. 1, with the main span length of 136 m, and locates in Fujian Province, southeast of China. It opened to traffic in January 1998. General layout of the bridge is shown in Fig. 2. The total length of the bridge is 166.84 m, and its width is 13.1 m.

The trussed arch rib section is 3.0 m deep and 1.6 m wide. The chord members use Φ550 mm × 8 mm steel pipes filled with C40 (concrete with compressive strength of 40MPa) and braces use Φ219 mm × 8 mm steel pipes, as shown in Fig. 3. The steel grade is Q235 (steel with yield stress of 235 MPa).

Fig. 1. *Photo of bridge.*

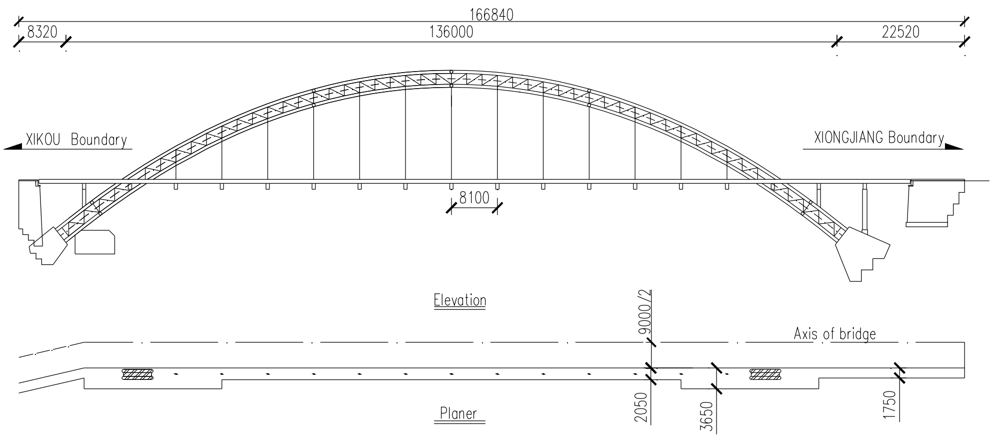


Fig. 2. *Elevation and planer view of bridge (Unit: mm).*

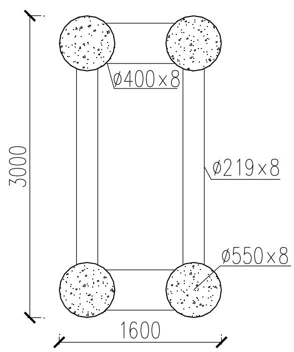


Fig. 3. *Cross section of main arch rib (Unit: mm).*

Fig. 4. Example of crack.

2.2. Cracks

In April 2013, cracks as shown in Fig. 4 were found in the connections of main arch chord and brace during the daily inspection. In total, there were nine cracks in the bridge, comprising eight in the left side of the main arch rib and one in the right side (the sides are determined by looking from Xikou boundary to Xiongjianng boundary). The distributions of cracks on left and right sides are shown in Fig. 5 and Fig. 6, respectively.

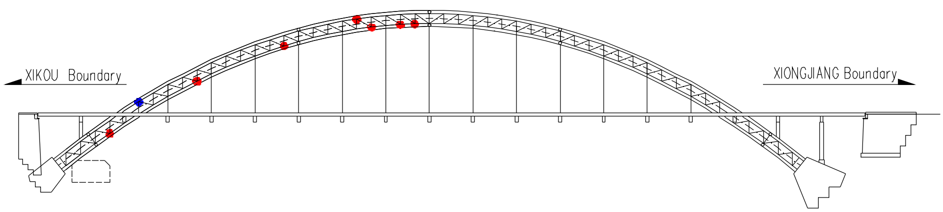


Fig. 5. Specific distribution of cracks in left side arch ribs.

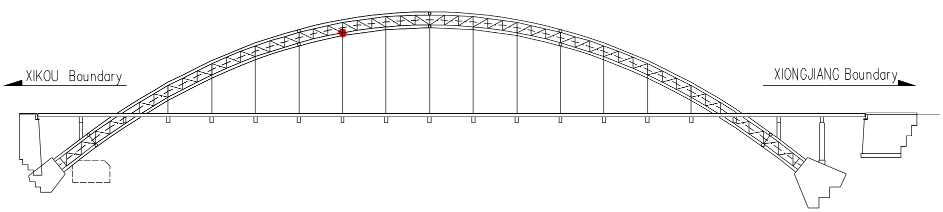


Fig. 6. Specific distribution of cracks in right side arch ribs.

3. FATIGUE LIFE ANALYSIS

3.1. Vehicle load and car model

The average daily traffic on the 5th, 15th, and 25th of each month was investigated from January 2010 to March 2013 except between October 2011 and February 2012 when the bridge was repaired and closed. Average daily traffic on this bridge reached 6000 to 8800. According to Tong's survey [11] on fatigue load spectrum for urban road bridges in Shanghai, vehicles over 9 ton may cause fatigue failure, which roughly correspond to tractors, large trucks, container vehicles in vehicle investigation. The number of such heavy vehicle has reached 1400 to 2600. If the truck is filled with sand or stone, its weight is approximately 38 ton or 42 ton, respectively. Therefore, the truck with the weight of 40 ton shown in Fig. 7 was used to calculate the fatigue damage.

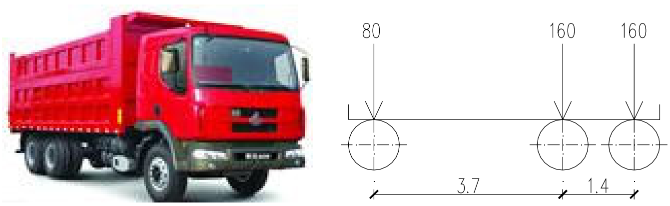


***Fig. 7.** Vehicle load (Unit:* m *and kN).*

3.2. Whole structure model and nominal stress

For joints with simple structure, nominal stress range is generally used for fatigue evaluation. Since the structures of the joints studied are complicated, it is very difficult to define the appropriate nominal stress range and corresponding S-N curve. Therefore, the HSS method is adopted. HSS is determined by extrapolation from the stress distribution approaching the weld, as shown in Fig. 8. It can be calculated by 1-point representative method and 2-points extrapolating method [12]. In both methods, very fine mesh is necessary. It causes huge number of nodes and elements if the whole structure model is used. So the whole structure model by beam element was used to get the axial force and bending moment near the joint, then local model of the joint was built to calculate the HSS with this axial force and bending moment.

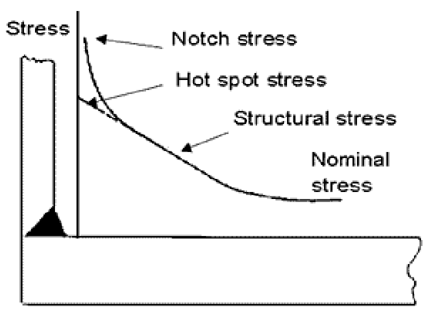


***Fig. 8.** Method of HSS.*

FE model of the whole structure was built by *midas Civil*. In total, there were 1302 nodes and 3462 elements in the model. The arch rib ends were fixed, one of the deck ends was pin-supported and the other end was roller-supported, as shown in Fig. 9. Two elements with the same nodes were used for a CFST chord to member consider the stiffness of steel and concrete parts of CFST independently. The position of vehicle load is shown in Fig. 10.

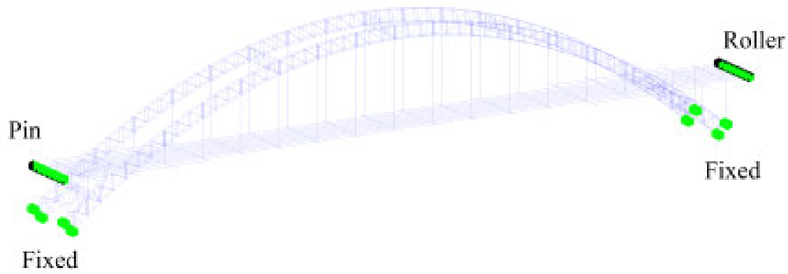


Fig. 9. *FE model by midas Civil.*

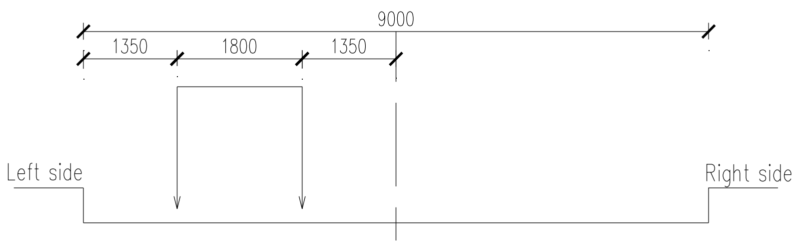


Fig. 10. *Vehicle layout (Unit:* mm*).*

Fatigue life of the CFST joint with the longest crack shown by a blue point in Fig. 5 was calculated. The shape of the joint is shown in Fig. 11. The axial forces and bending moment in chord tube and brace of this joint by the truck loading shown in Fig. 12 were calculated by the influence lines of this point. The bending moment in the brace was not considered since it was small enough.

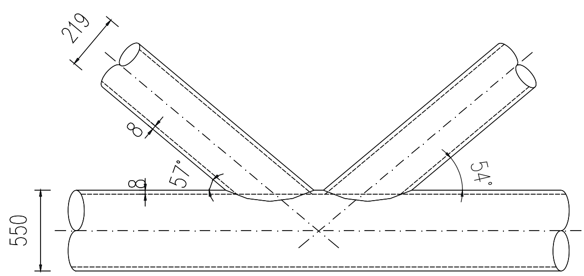
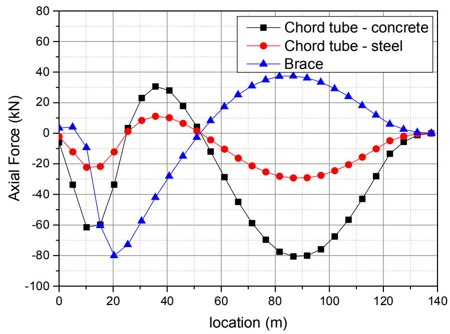
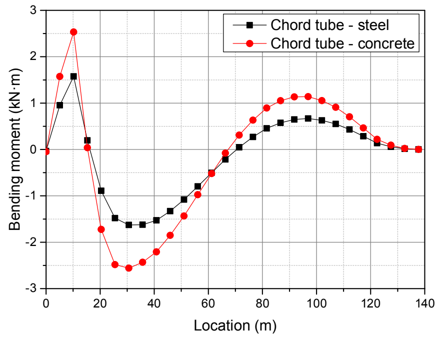


Fig. 11. *Shape of the CFST joint studied (Unit:* mm*).*





a) Axial force *b) Bending moment*

Fig. 12. *Axial force and bending moment of CFST joint.*

The nominal stresses calculated by equation (1) for chord tube and brace are shown in Fig. 13. It shows the stress ranges of chord tube and brace are 4.4 MPa and 22.2 MPa, respectively. The stress range of brace is much bigger than that of chord tube, so the loading positions inducing the minimum and maximum stress ranges in the brace were used to determine the axial forces and bending moments for the local model analysis. The locations of the vehicle for minimum and maximum stress ranges are also shown in Fig. 13, and the corresponding axial forces and bending moments are shown in Table 1.

$$\sigma = \frac{N}{A} + \frac{M}{I} y \tag{1}$$

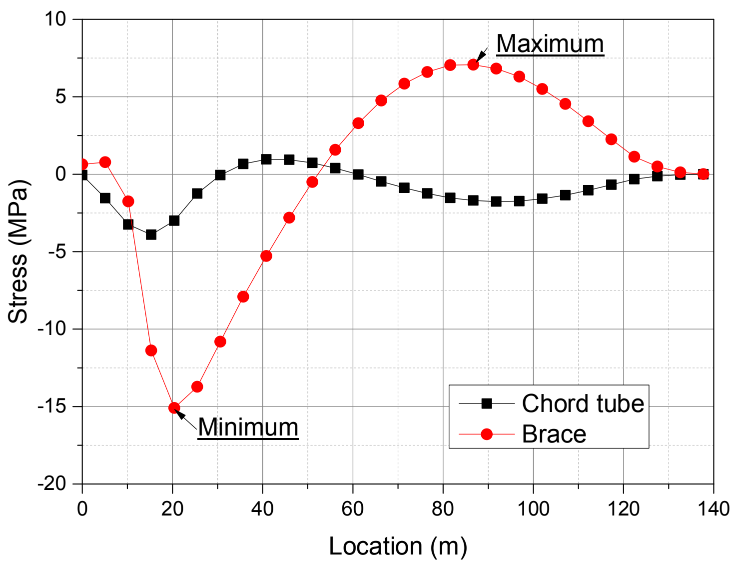


Fig. 13. *Nominal stress.*

Table 1. Axial force and bending moment for local model analysis.

Location		Minimum		Maximum	
		Left	Right	Left	Right
Chord	Axial force (kN)	-45.802	-136.174	-109.842	-69.861
	Bending moment (kN·m)	-2.613	-0.908	1.628	0.527
Brace	Axial force (kN)	-80.006	84.933	37.504	-36.365

3.3. Local model and HSS

The boundary condition of the model is shown in Fig. 14. The angle between the chord and right brace was modified as 57° and the left side axial force and bending moment in Table 1 were applied to the right brace for simplification. Geometric model of the joint was built by *3D CAD Autodesk Inventor*, then automatically meshed by *MSC.Apex*, and calculated by *MSC.Marc*. The local model is shown in Fig. 15. The lengths of chord tube and braces are 3000 mm and 2100 mm, respectively. The weld bead was not considered in this model, and the loading rigid plates were set at the ends of members to apply the axial forces and bending moments. The concrete and steel pipe were modeled independently, and integrated by the glue function. The quadrilateral shell element and hexahedral solid element were used for steel pipe and concrete, respectively. The 0.3 thickness method was adopted to determine the HSS around the intersecting line. Since the thickness of chord tube and brace is 8 mm, a mesh size near the intersecting line was set to 2.4 mm × 2.4 mm, then it became larger as 5 mm × 5 mm, 10 mm × 10 mm, 20 mm × 20 mm and 30 mm × 30 mm according to the distance from the intersecting line. The mesh size of concrete was 20mm × 20 mm. The loading rigid plate was meshed in size of 50 × 50 mm. The mechanical properties of CFST joint are given in Table 2.

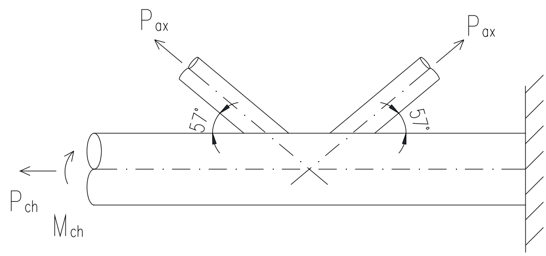


Fig. 14. Boundary condition.

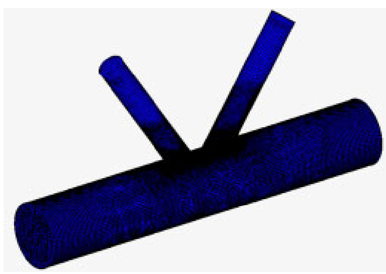


Fig. 15. Local FE model.

Table 2. Mechanical properties of CFST joint.

Material	Young modulus [MPa]	Poisson ratio
Steel	2.05×10^5	0.3
Concrete	3.25×10^4	0.2
Loading rigid plate	1.0×10^8	0.3

The stresses approximately perpendicular to the intersecting line of CHS and CFST joint by every 45° shown in Fig. 16 were picked up by *MSC.Marc*. They showed the stress concentration of CHS joint clearly, as shown in Fig. 17. The stresses obtained by the analysis for the CHS and CFST sections are shown in Fig. 18 and Fig. 19, respectively. It shows the peak SCF values of CHS are induced near 90° and 270°, while that of CFST is induced near 0°. The stress ranges of chord tube and brace in CHS and CFST joints are shown in Table 3. The maximum stress range is from 276.5 MPa to 51.1 MPa. It clearly shows the stress concentration becomes smaller when the chord tube is filled with concrete. CFST can be used to improve stress conditions, delay fatigue damage, and enhance fatigue life. Being different from nominal stress, the stress of chord tube is bigger than that of brace due to the large out-plane bending acting on the chord tube. The nominal stress range is 22.2 MPa, so the SCFs of CFST joint are between 1.2 and 2.3 around the intersecting line.

The Young's modulus of concrete used for this bridge is 3.25×10^4 MPa, and that of C15 (concrete with compressive strength of 15 MPa) is 2.20×10^4 MPa. It was assumed that the Young's modulus became lower due to deterioration, and the value of 2.20×10^4 MPa was used in the calculation. The stress ranges are shown in Table 4. It shows the stress range becomes larger when Young's modulus becomes smaller.

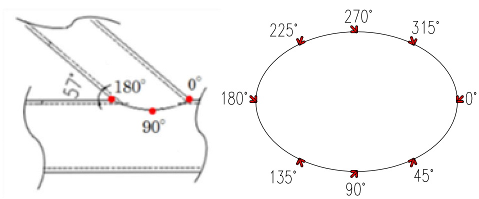


Fig. 16. Position of stress output.

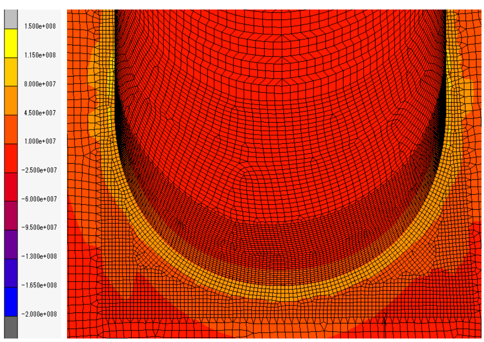


Fig. 17. Stress cloud of CHS.

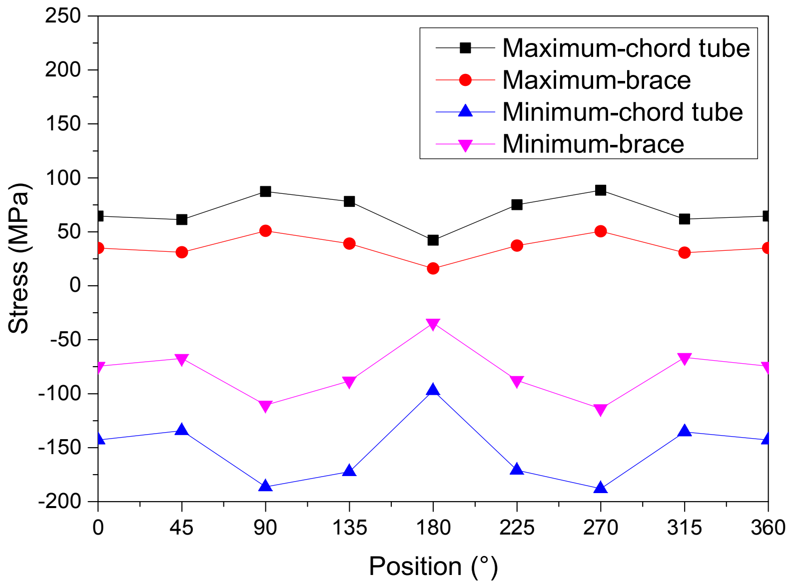


Fig. 18. *Stress distribution of CHS.*

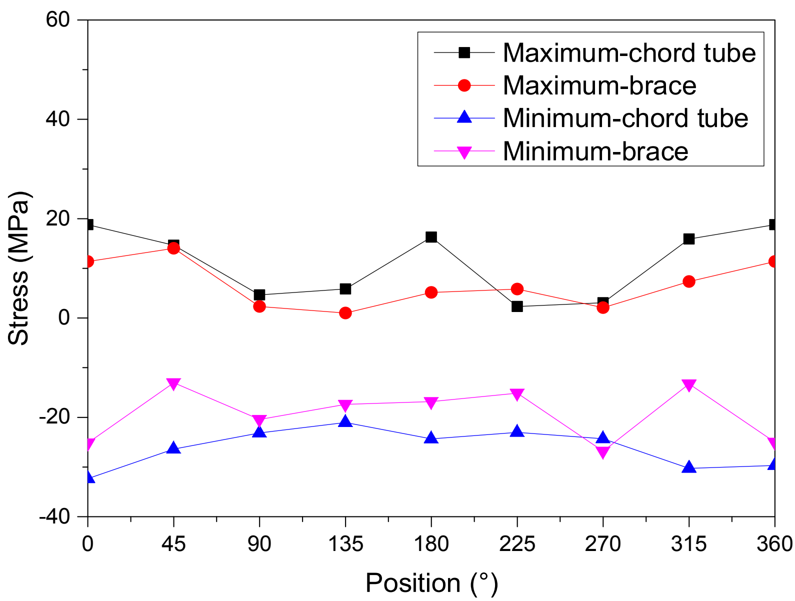


Fig. 19. *Stress distribution of CFST.*

Table 3. Stress ranges at different position of CHS and CFST joint (Unit: MPa).

Position	CHS		CFST	
	Chord	Brace	Chord	Brace
0°	207.44	109.48	51.13	36.53
45°	195.51	98.34	41.02	27.06
90°	273.56	161.50	27.80	22.71
135°	250.36	127.33	26.88	18.39
180°	139.41	50.57	40.61	21.97
225°	246.04	124.98	25.33	20.95
270°	276.49	164.45	27.36	28.91
315°	197.21	97.00	46.16	20.61

Table 4. Stress range at different position of the CFST joint (Unit: MPa).

Position	Original		Low Young's modulus	
	Chord	Brace	Chord	Brace
0°	51.13	36.53	54.63	38.02
45°	41.02	27.06	46.53	37.10
90°	27.80	22.71	26.48	21.63
135°	26.88	18.39	34.83	15.03
180°	40.61	21.97	43.20	23.39
225°	25.33	20.95	29.00	15.04
270°	27.36	28.91	32.26	26.13
315°	46.16	20.61	49.77	28.07

3.4. Fatigue life

In this bridge, the crack was found near 180°, so the fatigue life of this point on chord tube was calculated. The S-N curves by Comité International pour le development et l'étude de la construction tubulaire (CIDECT) [13] and Japanese Society of Steel Construction (JSSC) [14] were used in this study, as shown in Fig. 20.

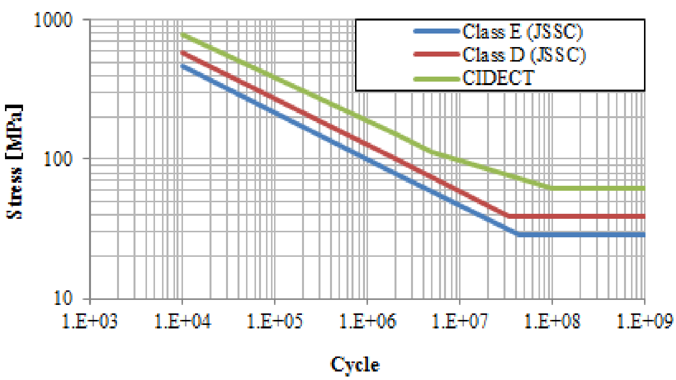


Fig. 20. S-N curves of CIDECT and JSSC.

By using the HSS obtained with the local model FE analyses of CHS and CFST joints, the number of cycles to failure and fatigue life was estimated. The results are shown in Table 4 and Table 5, respectively. Because the cut-off limit is 61 MPa in CIDECT, the CFST joint has infinite life under CIDECT, while the fatigue life under JSSC (class D) is 58.7 years and 31.6 years for 1400 and 2600 trucks/day, respectively. Fatigue life under JSSC (class E) is 31.3 years and 16.9 years for 1400 and 2600 trucks/day, respectively. When the average daily traffic is 2600, the fatigue life calculated by class E (JSSC) is the closest to actual situation. Compared with class D (JSSC), the fatigue life of class E (JSSC) decreases to half level.

Table 4. *Number of cycles to failure.*

Section	HSS ranges [MPa]	Cycles to failure		
		CIDECT	**JSSC (class D)**	**JSSC (class E)**
CHS	139.41	2.5×10^6	9.0×10^5	4.5×10^5
CFST	40.61	/	3.0×10^7	1.6×10^7
CFST(Young's modulus)	43.20	/	2.5×10^7	1.3×10^7

Table 5. *Fatigue life (Unit:* Year*).*

Section	CIDECT		JSSC (class D)		JSSC (class E)	
	1400 trucks /day	**2600 trucks /day**	**1400 trucks /day**	**2600 trucks /day**	**1400 trucks /day**	**2600 trucks /day**
CHS	4.9	2.6	1.8	0.9	0.9	0.5
CFST	/	/	58.7	31.6	31.3	16.9
CFST(Young's modulus)	/	/	48.9	26.3	25.4	13.7

4. LAST REMARKS

In this study, numerical analyses of SCFs of the K-joint with the longest crack in an existing bridge were conducted by *MSC.Marc*. The stress distributions along the weld toe and hot spot stress of the K-joint were obtained. The main conclusions can be summarized as follows.

(1) In this bridge, when the average daily traffic is 2600, the fatigue life calculated by class E (JSSC) is 16.9 years, which is the closest to actual situation.

(2) The SCFs of CFST joint in this bridge are between 1.2 and 2.3 around the intersecting line. It clearly shows the stress concentration becomes smaller when the chord tube is filled with concrete. CFST can be used to improve stress conditions, delay fatigue damage, and enhance fatigue life.

(3) The stress range becomes larger with the decrease in Young's modulus, so the strength of concrete should be ensured.

(4) Comparing CIDECT and JSSC, the estimated fatigue lives are quite different.

Since the accuracy of the fatigue life estimated in this study is not clear, the research should be going on. Some ideas to be done in the future work can be listed as follows.

(1) The interface between concrete and steel should be considered more accurately, and the non-linear material property of concrete also should be considered.

(2) Static loading test and fatigue test of CFST joint models will be conducted to reveal the general tendency of crack initiation and growth.

(3) S-N curves and fatigue calculation method of CFST joint will be proposed.

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