

Fatigue Damage of a Half-through Concrete-filled Steel Tubular Trussed Arch Bridge in China

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ABSTRACT

Concrete-filled steel tubular (CFST) trussed structure, adopting relatively small pipes mainly subjected to axial force to achieve greater vertical and transverse stiffness, is a kind of structure suitable for long-span bridges. In China, CFST arch bridges have been rapidly developed since 1990. Until now more than 400 CFST arch bridges with the span length over 50 meters have been built. Among them, trussed arch rib accounts for 39% of the total number, and 59% of the total number has the span length over 100 meters. Since the history of CFST arch bridge construction is short, the present specifications only provide some structural types to prevent fatigue. No provisions are prepared for fatigue design. In April 2013, cracks in the connections of arch chord and web member of a half-through CFST trussed arch bridge with the main span of 136 meters were found during the daily inspection, which seriously damage the safety of the bridge. In this study, based on the information of this bridge and its damages obtained from website survey and literature reviews, main cause of the cracks is identified and fatigue life was estimated by finite element analysis and hot spot stress method. Furthermore, an improved design of the bridge was suggested.

1 Introduction

CFST arch bridges have become an important bridge type in China. No.1 Hejiang Yangtze River Bridge with the span length of 530 meters is the longest CFST arch bridge in the world, which was built in 2013. CFST is expected to improve stress conditions, delay fatigue damage, and enhance fatigue life.

For full penetration butt welds of CFST joint, the axial stiffness of web member is greater than the radial stiffness of chord tube. The intersecting line becomes the weak part of the whole structure, which leads to stress concentration and weld defects (Ju 2008, Bai 2010). Furthermore, during welding process of CFST joint, it is easy to produce uneven temperature field, and the self-equilibrated residual stress in vertical and lateral directions usually reaches the yield point of the material, which aggravates the fatigue issue (Wu 2010). By comparing fatigue fracture behavior and fatigue life of CFST and hollow tube joint, Fan (2006, 2007, 2009) creatively proposed and confirmed

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correction coefficient method, experience method and natural flaw method, which were used to predict fatigue life of CFST joint. In correction coefficient method, it calculated the fatigue life of CFST joint by correction coefficient, which reflected the improvement of fatigue life of the hollow tube joint. In experience method and natural flaw method, based on the fracture mechanics, an initial crack was assumed to calculate the fatigue life of CFST joint. Based on static test and fatigue test of welded CFST T-joints, Zhu (2007) created S-N curve about the welded CFST T-joints, which was obtained by linear regression analysis. Sakai (2004) 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 resistance of the concrete filled specimen was about twice that of the concrete non-filled specimen. However, in order to reduce localized stress at the weld toe of the joint, reinforcement was needed. Based on fracture and damage mechanics, Zhang (2012) analyzes the fatigue life of CFST arch bridge, and concluded that cycles to failure, stress ranges and form of joints should be comprehensively considered. By using commercial FE software, Yuan (2000), Chen (2007) and Yan (2007) analyzed stress concentration factor and fatigue life of CFST arch rib in Yajisha Bridge, Maocaojie Bridge and Haikou Bridge in China, respectively. Haldimann (2008) 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. However, results of these studied have not been reflected in specifications yet.

At the present time, S-N curve and provisions for fatigue checking calculation of CFST joints are not given by the current specifications due to lack of experimental verification. The specifications only provide some structural types to prevent fatigue.

In this study, main cause of the cracks found in a half-through CFST arch bridge during the daily inspection in April 2013 was identified. Fatigue life was estimated by FE analysis and hot spot stress (HSS) method. Furthermore, an improved design of the bridge was suggested.

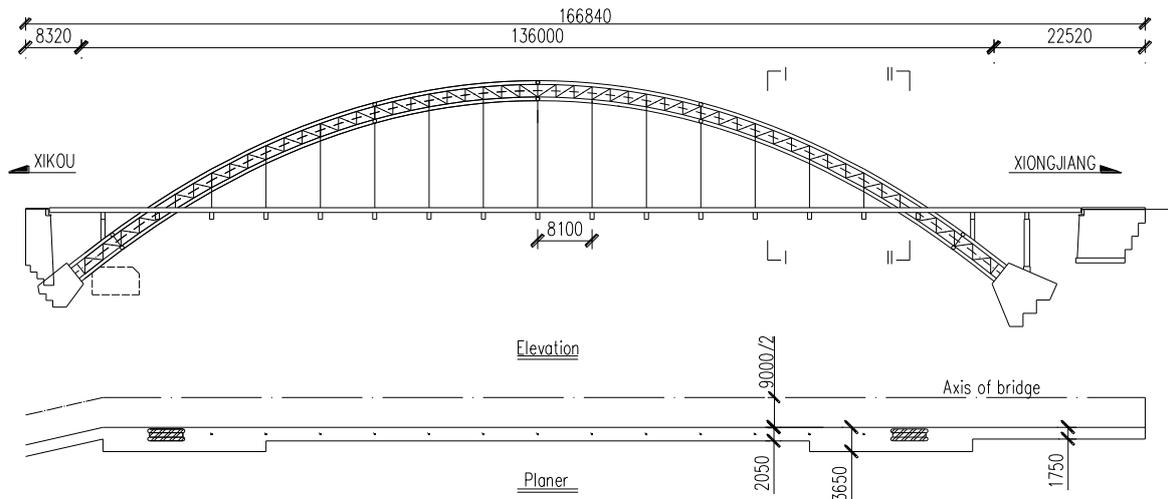
2 Outline of bridge and cracks

2.1 Bridge

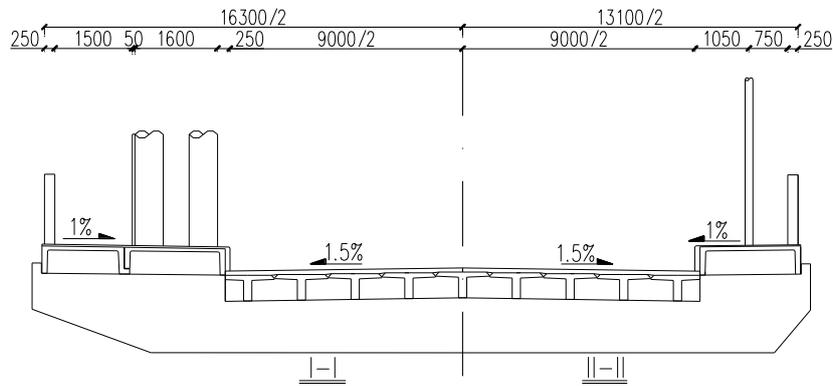
This bridge is a half-through CFST trussed arch bridge in Fujian Province, China, as shown in Fig. 1. The length of the bridge is 166.84 meters, and its main span is 136 meters. General layout of the bridge is shown in Fig. 2. It opened to traffic in January 1998.



Fig. 1 Photo of bridge



(a) Elevation and planer view



(b) Side view

Fig. 2 Overall layout drawing of bridge(Unit: mm)

The trussed arch rib section is 3.0 meters deep and 1.6 meters wide. The chord tubes use $\Phi 550\text{mm} \times 8\text{mm}$ steel pipes filled with C40 (concrete with compressive strength of 40MPa) and web members use $\Phi 219\text{mm} \times 8\text{mm}$ steel pipes, as shown in Fig. 3. The steel grade is Q235 (steel with yield stress of 235MPa).

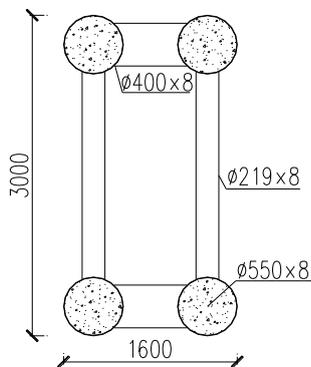


Fig. 3 Cross sections of main arch rib(Unit: mm)



Fig. 4 Example of crack

2.2 Cracks

In April 2013, cracks were found in the connections of main arch chord and web member during the daily inspection, as shown in Fig. 4. In total, there are nine cracks in the bridge, comprising eight in the left side of the main arch rib and one in the right side (from XIKOU to XIONGJIANNG, left hand is left side and right hand is right side). The notation of arch chord is shown in Fig. 5. The position and length of cracks are summarized in Table 1. Specific distribution of cracks is shown in Fig. 6 and Fig. 7.



Fig. 5 Notation of arch chord

Table 1 Summary of cracks (Unit: mm)

No.	Position	Length
1#	Left, between 3'# and 4'# suspender, Left, 11'#tilted web member and arch chord C	220
5#	Left, between 5'# and 6'# suspender, Left, 17'#tilted web member and arch chord C	200
6#	Left, between 6'# and 7'# suspender, Left, 20'#tilted web member and arch chord A	160
8#	Left, between 7'# and 8'# suspender, Left, 23'#tilted web member and arch chord C	220
35#	Right, between 2'# and 3'# suspender, Left, 7'#tilted web member and arch chord H	180
46#	Left, between 0'# and 1'# suspender, Left, 2'#tilted web member and arch chord D	200
47#	Left, between 0'# and 1'# suspender, Right, 2'#Vertical web member and arch chord D	250
49#	Left, between 1'# and 2'# suspender, Left, 5'#tilted web member and arch chord C	170
49'#	Left, between 1'# and 2'# suspender, Left, 5'#tilted web member and arch chord A	200

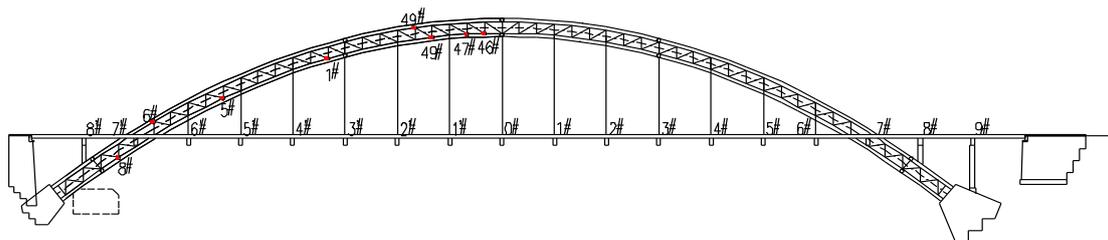


Fig. 6 Specific distribution welded cracking of left side

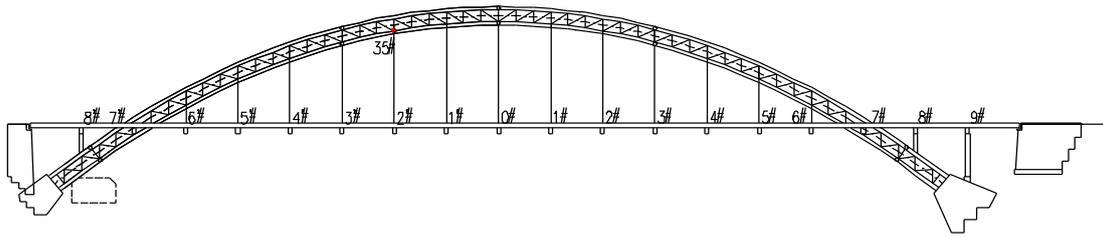


Fig. 7 Specific distribution of welded cracking of right side

2.3 Evaluation of load bearing capacity

This bridge had not had longitudinal beam. In 2007, a steel stiffening truss was added to the bridge. In 2012, all suspenders were replaced. After that, static and dynamic test was done. It showed that: stiffness and strength met the design requirements, but the bridge seriously vibrated. The load bearing capacity of the trussed arch rib under the condition of partial web member failure was analyzed. FE model was built by *MIDAS/CIVIL* and safety check on the components of trussed arch rib and the whole structure was carried out according to *Technical Code for Concrete-filled Steel Tube Arch Bridges* (GB50923-2013). Their results showed that both of the main arch cross section strength and the overall stability of the main arch met the requirements under the condition of partial web member fracture, and trussed arch rib was still able to withstand the design load even when partial web member failed.

2.4 Weld quality

It used unilateral groove weld with the groove depth of 6mm. Through the magnetic particle testing and ultrasonic testing, no weld defects such as crack, flash welding seam, burn through, arc pit, porosity, slag inclusion, bite edge, incomplete fusion, and lack of penetration were found. So the welding quality problem is excluded from the cause of cracking.

3 Traffic condition

As shown in Table 2, according to *Technical Standard of Highway Engineering* (JTG B01-2003) and the data from Fujian highway administrative bureau, the vehicle can be divided into small vehicle, large vehicle, small truck, middle-sized truck, large truck, trailer, super large truck, container car and tractor. Different vehicle uses different space occupancy as heavy vehicle takes more space. In order to take such influence, the traffic volume convert coefficient from 1.0 to 4.0 is specified for each vehicle type based on boarding capacity or loading capacity.

On the 5th, 15th, 25th of each month from January 2010 to March 2013, the average daily traffic was investigated (The bridge was repaired between October 2011 and February 2012, and stopped to use), as shown in Table 3. Average daily traffic has reached 6000 to 8800. According to Tong's(1997) survey 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, which accounted for 36% to 48% of the total. Since approximately half of them were empty cars. Daily traffic which may cause fatigue was about 21% and year's traffic growth rate reached 5%, which is

greater than the average level of 3% in Fujian Province. It aggravated fatigue issue.

Table 2 Vehicle classification and traffic volume convert coefficients

Vehicle classification	Traffic volume convert coefficients	Explanation
small vehicle(SV)	1.0	Boarding capacity ≤ 19
large vehicle(LV)	1.5	Boarding capacity > 19
small truck(ST)	1.0	Loading capacity $\leq 2\text{ton}$
middle-sized truck(MT)	1.5	$2\text{ton} < \text{Loading capacity} \leq 7\text{ton}$
large truck(LT)	3.0	$7\text{ton} < \text{Loading capacity} \leq 14\text{ton}$
Trailer	3.0	-
super large truck(SLT)	4.0	Loading capacity $> 14\text{ton}$
container car(CC)	4.0	-
Tractor	4.0	-

Table 3 Average daily traffic of the bridge

Quarter	ST	MT	LT	SV	LV	Trailer	Tractor	SLT	CC	Convert total	Growth rate	
2010	1st	461	508	477	1703	277	74	125	37	128	6229	-
	2nd	503	435	331	2134	265	83	78	98	112	6164	-1%
	3rd	567	574	517	2242	275	151	126	148	167	8002	30%
	4th	716	634	576	2005	323	174	112	73	158	7953	-1%
2011	1st	606	570	357	2869	258	138	83	85	167	7680	-3%
	2nd	666	654	591	2213	325	0	57	443	145	8701	13%
	3rd	598	486	441	2136	281	0	64	291	106	8570	-2%
2012	2nd	661	606	512	2391	249	0	43	347	96	7052	-
	3rd	574	515	612	2726	306	0	68	328	59	7815	11%
	4th	679	488	340	2835	259	0	68	289	88	8188	5%
2013	1st	598	486	441	2136	281	0	64	291	106	7435	-9%

4 Fatigue life analysis

4.1 FE model and influence line

FE model was built by *MIDAS/CIVIL*. In total, there were 1302 nodes and 3462 elements in the model, as shown in Fig. 8. Continuous chords and pin-ended braces are assumed as described in a reference (Zhao 2001). Two elements with the same nodes are used for a CFST chord member to consider the stiffness of steel and concrete part of CFST independently.

According to vehicle Investigation, if the truck is filled with sand, its weight may be 38 ton; and if the truck is filled with stock, its weight may be 42 ton. As shown in Fig. 9, the truck with the weight of 40 ton was used to calculate the fatigue damage. The position of vehicle load is shown in Fig. 10.

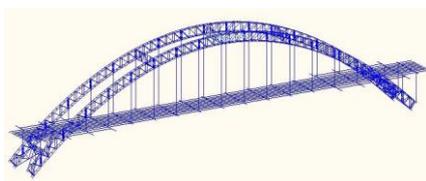


Fig. 8 FE model

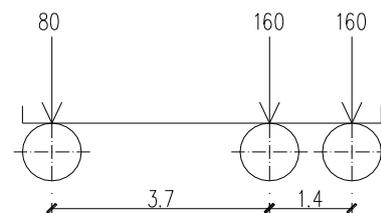


Fig. 9 Vehicle load (Unit: m and kN)

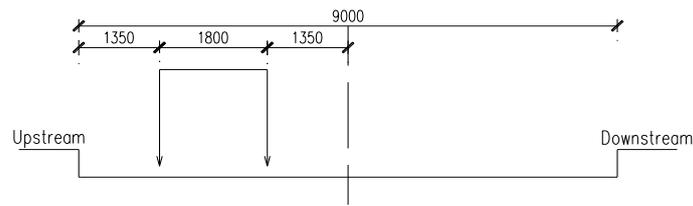
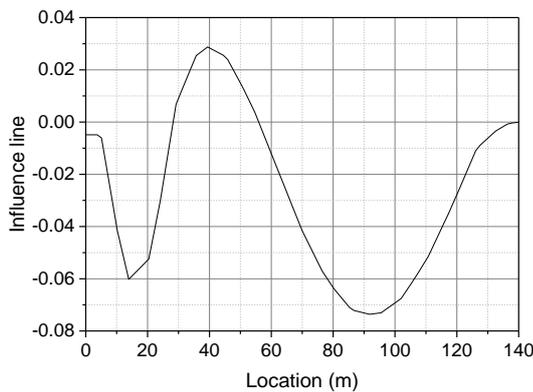
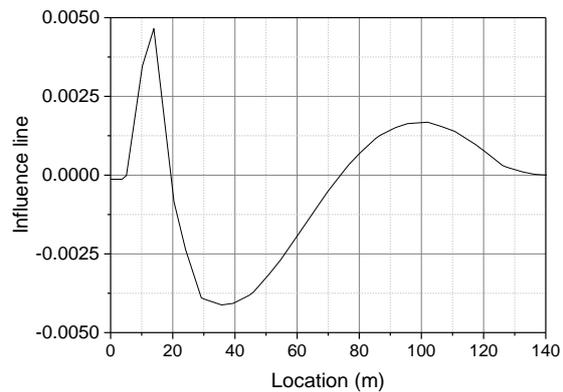


Fig. 10 Vehicle layout (Unit: mm)

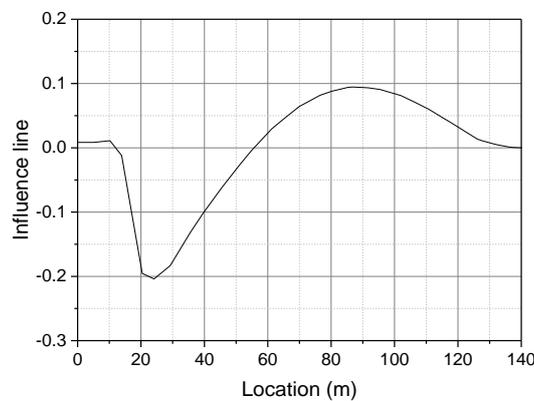
Fatigue life of the 6# crack of CFST joint was calculated. The influence lines of axial forces and bending moment in chord tube and web member at the joint are shown in Fig. 11.



(a) Axial force of chord tube



(b) Bending moment of chord tube



(c) Axial force of web member

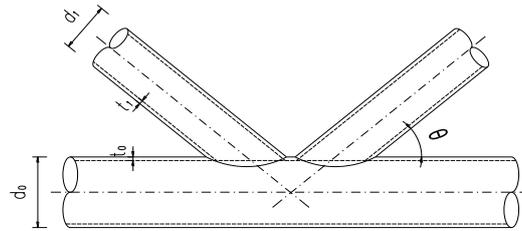
Fig. 11 Influence lines

4.2 Fatigue life

Provisions on fatigue design of hollow tube structures can be found in many design codes or specifications, such as American Petroleum Institute (API), American Welding Society (AWS), International Institute of Welding (IIW), European Convention for Constructional Steelwork (ECCS), etc. However, due to the lack of experimental verification, S-N curve and provisions for fatigue checking calculation are not given by the current specification, but only some structural types to prevent fatigue. Therefore, a fatigue life estimation method for hollow tube structure in the reference (Zhao 2001) is used in this study.

Range of structural parameters valid for the method is shown in Fig. 12. It should be noted that the diameter-thickness ratio of this CFST arch bridge is 34.4, which is a little larger than applicable range of the design code. Nevertheless, the code was applied since the estimated fatigue life by the method will be underestimated due to the fatigue performance improvement by CFST.

The axial forces and bending moments in the joint can be treated as a combination of condition 1 (basic balanced axial loading) and condition 2 (chord loading), as shown in Fig. 13.



Range of validity: diameter ratio: $0.30 \leq \beta \leq 0.60$, diameter-thickness ratio: $24 \leq 2\gamma \leq 60$, thickness ratio: $0.25 \leq \tau \leq 1.00$, angel: $30^\circ \leq \theta \leq 60^\circ$.

Fig. 12 Range of structural parameters valid for the method

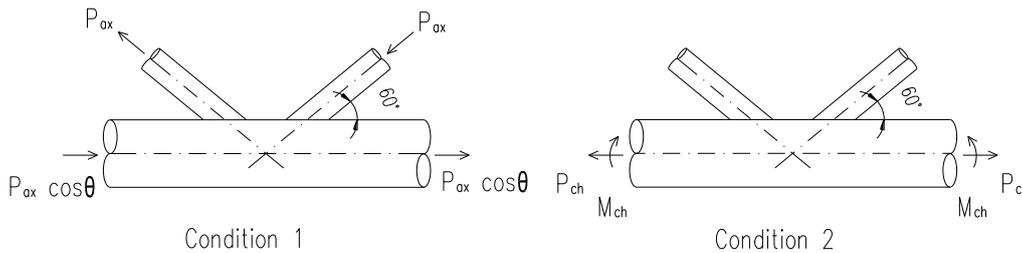


Fig. 13 Two basic fatigue load cases

In order to take into account the effects of the secondary bending moments, nominal stress ranges have to be multiplied by magnification factors, which are given by EN 1993-1-9. So,

$$\sigma_{chord,ch} = \sigma_{chord,ax} + \sigma_{chord,ipd} = MF \cdot \frac{P_{ch}}{A} + \frac{M_{ch}}{w} \text{ and } MF_{chord} = 1.5$$

$$\sigma_{brace,ax} = MF \cdot \frac{P_{ax}}{A} \text{ and } MF_{brace} = 1.3$$

The design values of the nominal stress for chord and brace are shown in Fig. 14.

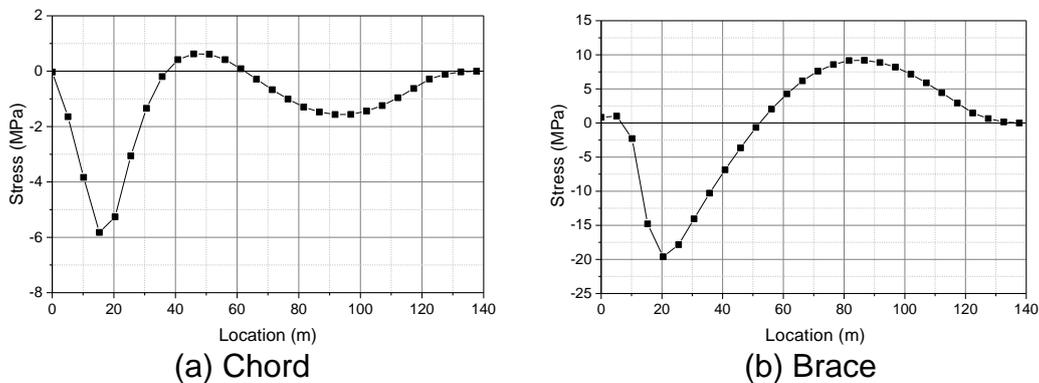


Fig. 14 Nominal stress (Unit: MPa)

Stress concentration factors (SCF) calculation for condition 1:

$$SCF_{ch,ax} = \left[\frac{Y}{12}\right]^{0.4} \left[\frac{T}{0.5}\right]^{1.1} \cdot SCF_{o,ch,ax} \quad \text{and} \quad SCF_{b,ax} = \left[\frac{Y}{12}\right]^{0.5} \left[\frac{T}{0.5}\right]^{0.5} \cdot SCF_{o,b,ax}$$

SCF calculation for condition 2:

$$SCF_{ch,ch} = 1.2 \times \left[\frac{T}{0.5}\right]^{0.3} \cdot (\sin \theta)^{-0.9} \quad \text{and minimum } SCF_{ch,ch} \text{ of 2.0 is adopted.}$$

$$SCF_{b,ch} = 0$$

A partial safety factor for fatigue loading and fatigue strength needs to be taken into consideration for design. However, the partial safety factor is not considered because the aim of this study is just fatigue life estimation of an existing bridge. HSS ranges calculated by the following equations are shown in Fig. 15.

$$S_{rhs, chord} = SCF_{ch,ax} \sigma_{brace,ax} + SCF_{ch,ch} \sigma_{brace,ch} \quad \text{and} \quad S_{rhs, brace} = SCF_{b,ax} \sigma_{brace,ax}$$

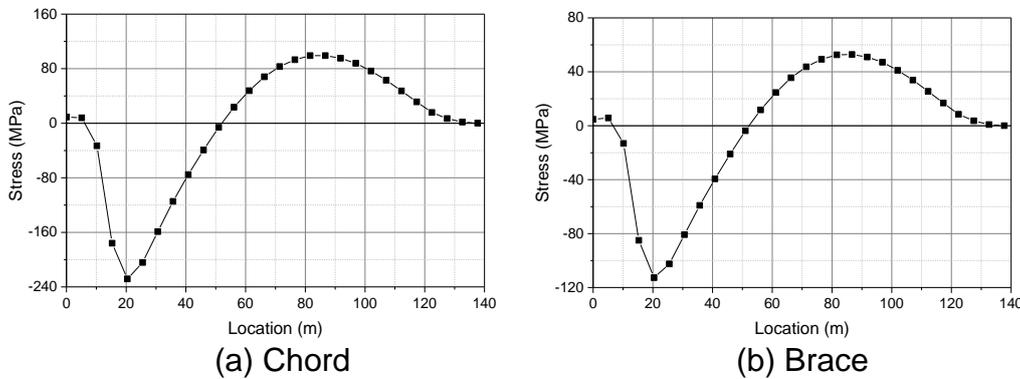


Fig. 15 HSS (Unit: MPa)

A common set of $S_{rhs} - N_f$ curves and thickness correction formulae have been established (Zhao 2001). Based on the analysis in the data sets of circular hollow section, the number of cycle to failure is:

$$\text{For } 10^3 < N_f < 5 \cdot 10^6, \quad \log(N) = \frac{12.475 - 3 \cdot \log(S)}{1 - 0.18 \cdot \log\left(\frac{16}{t}\right)}$$

$$\text{For } 5 \cdot 10^6 < N_f < 10^8, \quad \log(N) = 16.327 - 5 \cdot \log(S) + 2.01 \cdot \log\left(\frac{16}{t}\right)$$

So, SCF and number of cycles to failure according to HSS method and correction coefficient method obtained for these two conditions are shown in Table 4. Considering the traffic volume on the bridge, the calculated number of cycles to failure roughly corresponds to 7 years.

Table 4 Number of cycles to failure

SCF				HSS ranges (Unit: MPa)		Cycles to failure	
condition 1		condition 2		chord	brace	chord	brace
chord	brace	chord	brace	chord	brace	chord	brace
11.1	5.7	2.0	0	327.505	165.543	5.3×10^5	4.1×10^6

4.3 Improved design

By analyzing the stress ranges under different parameters, we obtain the main parameters affecting fatigue life: diameter ratio, thickness ratio and diameter-thickness ratio. When diameter ratio of trussed arch rib change from 1:2.5 to 1:1.67 and diameter-thickness ratio change from 68.75 to 47, the parameter of diameter and thickness will change as shown in Table 5.

Table 5 Parameter of diameter and thickness

	Original Design	Improved Design
Chord Tube Diameter	550mm	750mm
Web Member Diameter	219mm	450mm
Chord Tube Thickness	8mm	16mm
Web Member Thickness	8mm	8mm

It results in the reduction of SCF and HSS ranges as shown in Table 6. When thickness is 16mm, HSS ranges are lower than the constant amplitude fatigue limit of 84N/mm^2 listed in the reference (Zhao 2001). Therefore no fatigue damage is expected to occur.

Table 6 SCF and HSS ranges of original and improved design

	SCF				HSS ranges (Unit: MPa)	
	chord		brace		chord	brace
	chord	brace	chord	brace		
Original	11.1	5.7	2.0	0	327.505	165.543
Improved	4.1	2.8	2.0	0	57.252	37.103

5 Last remarks

In this study, main cause of the cracks found in a half-through CFST arch bridge in China was identified by fatigue life estimation with FE analysis and HSS method. Furthermore, an improved design of the bridge was suggested. Main conclusions can be summarized as follows.

(1) Because the average daily traffic of the bridge has reached 7435 in 2013, and overweight vehicles were more than expectation, we had reasons to believe that the main cause of the cracking should be fatigue by the vehicle loads.

(2) Considering the traffic volume on the bridge, the calculated number of cycles to failure of the 6# crack of CFST roughly corresponds joint is to about 7 years.

(3) When diameter ratio of bridge truss arch rib adjusted from 1:2.5 to 1:1.67 and diameter-thickness ratio changed from 68.75 to 47, the HSS range is lower than the constant amplitude fatigue limit. Therefore no fatigue damage is expected to occur.

In this study, the method originally developed for hollow tube structures was adopted. However, applicability of the method to CFST is not clear. Some ideas to be done in the future work can be listed as follows.

(1) FE analysis of local models will be built. Stress distribution, SCF and HSS of chord and web member adjacent to intersecting line will be revealed.

(2) Static loading test and fatigue test of CFST joint models will be conducted. The

general tendency of crack initiation and growth will be revealed.

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

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