불량용접을 갖는 트러스교의 피로수명

Expectation of the Fatigue Life at the Truss Bridge Including Improper Welding

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Abstract

수많은 판형부재로 구성된 대형 트러스교에서는 불량용접된 부재가 존재할 수 있으며, 이는 교량의 피로수명에 결정적인 영향을 줄 수 있다. 이러한 교량에서 용접불량부를 조사하고 부재단면중 용접불량을 고려한 유효단면을 가정하여 피로수명을 예측하는 방법에 대하여 연구하였다. 이를 위하여 피로수명에 영향을 미치는 교통량을 분석하고 차량모델을 가정하여 유효등가응력을 산정하였으며 모형피로시험에서 구한 응력-반복횟수 곡선을 이용하여 피로수명을 예측하였다. 본 연구에서 사용된 분석기법은 불량용접부와 교통량이 강교량의 피로수명에 미치는 영향을 예측하는데 매우 유용함을 알 수 있다.

Key words : Fatigue, Service Life, Improper Welding, Average Daily Traffic(ADT), Equivalent Stress Range, Truss Bridge

1. Introduction

The rapid economic development in the 1970s necessitated massive investment in the development of social infrastructure, notably in nationwide road network. New expressways were constructed, existing national roads were expanded and road network in major cities were readjusted, all leading to the construction of numerous bridges. However, as the nation had to build many roads and bridges in a relatively short period of time, construction of such structures had to become economy-driven in many cases and this led to neglected endurance and safety of the bridges in some cases. Therefore, starting mid-1980, safety-related problems of some structures that had been built in the economy-driven 1970s began to emerge on and off and in 1990s such problems increased in terms of frequency. One of serious problems is the fatigue behavior of steel bridges under the increasing traffic volume. The other study

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on fatigue load models showed that load spectra were site specific and stress spectra were component specific.[1] The improperly welded structural members composed of steel plates severely affected the fatigue behavior of steel bridges and caused to the shortening of fatigue life. This study is intended to analyze the fatigue effect of improper welding considering the traffic volume and to estimate the life span on the bridge with improperly welded members.

2. Improper welding in the vertical member of truss bridge

The model bridge in this study is shown in Fig. 1, which is five-span continuous gerber truss bridge with the height of the truss varying along the span. The suspended truss was hinge-connected with anchor trusses at both ends. The design load is DL-18[2], which is not classified as the second class bridge but as the first class bridge at the construction time.

All the members in truss bridge were not rolled beams but plate girders which were manufactured by welding at the intersection line of steel plates. Especially vertical members had welding connections in the axial direction because of the stress concentration around the pin joint at the both ends. It was found that the welding between the upper thick plate and the lower thin plate was so improperly done shown as Fig. 2.

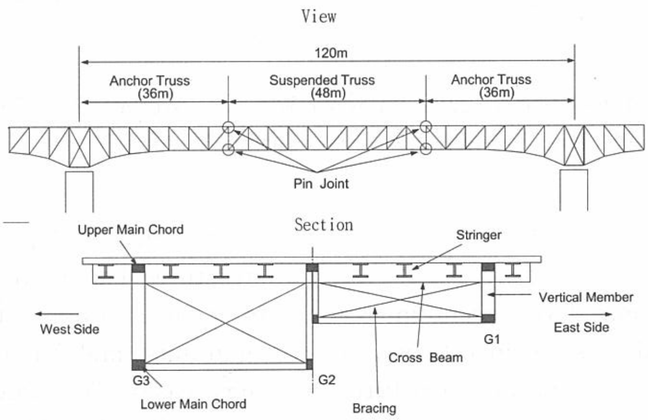
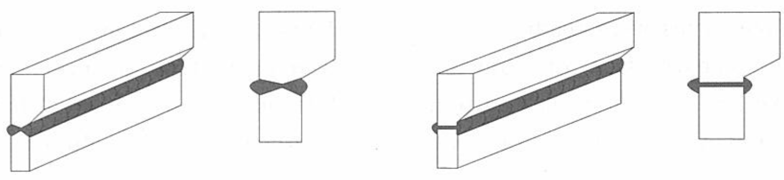


Fig. 1 Structural geometry and dimensions



(a) At the time of the design (b) At the time of the construction

Fig. 2 Improperly welded plate girders

This may affect the structural safety and the fatigue life of the bridge, so analytical method and experimental test are needed to verify the effect of improper welding. In this study analytical method was introduced referring the experimental results conducted at the laboratory[3].

3. Structural analysis considering the section loss due to improper welding

The welding section between the upper plate and the lower plate in the vertical member is not done in groove welding method as designed but in fillet welding method. The section was not welded all over the plane of the H-shaped plate girder. Consequently, the effective section area is substantially reduced from the one specified in the design. The effective section area of the vertical member at the welding joint is measured and evaluated. Supposed effective section areas for the analysis are shown in Fig. 3. The supposed cases are 1)the section area of the vertical member in the design, 2) three assumed section areas reduced from improper welding in the initial stage, 3) the status of the vertical member at the time of initial crack and before collapse.

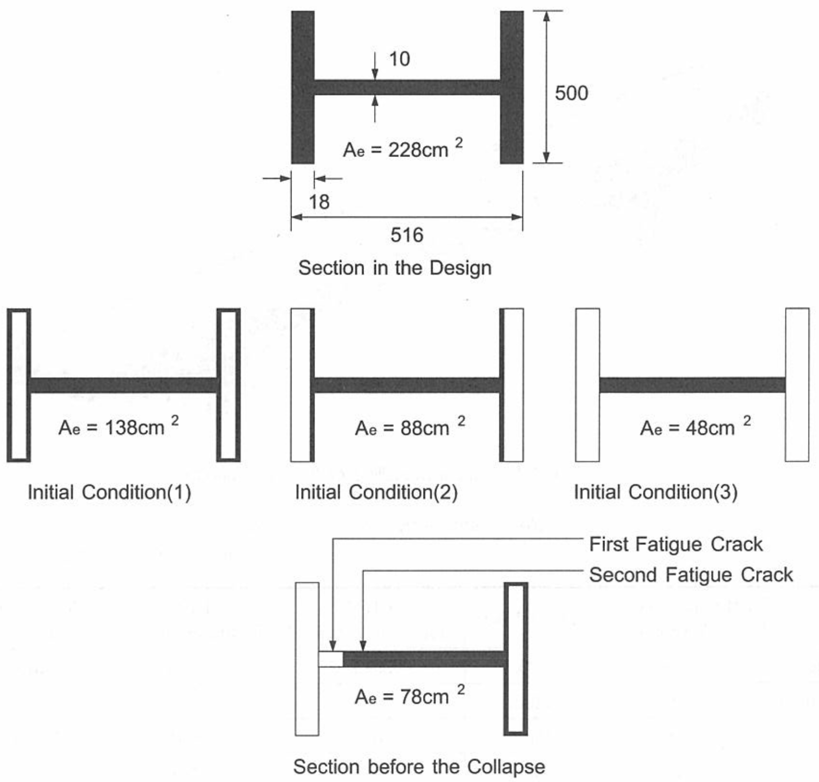


Fig. 3 Supposed effective section areas in various conditions

Fig. 4 showed the numerical model which included one suspended span and two anchor spans. The load effect of connected span was considered as the point load. The results of analysis was shown as Table 1. This results shows that analytical values of the original design were satisfactory under both the desin load DL18 and the DL24, but those of some cases with improper weldings were beyond allowable values specified in the design code under DL18. In some cases stress level reached the yield point or the fracture under the DL18. This results means that the section loss due to improper welding caused the vertical member of the bridge to yielding or fracture under the design load.

4. Fatigue life analysis by the traffic survey

4.1 Average daily traffic

The truck loads may affect the fatigue life of the steel bridge. The traffic survey[4] was performed to obtain the equivalent stress range from the different types of vehicles having variable total weights.

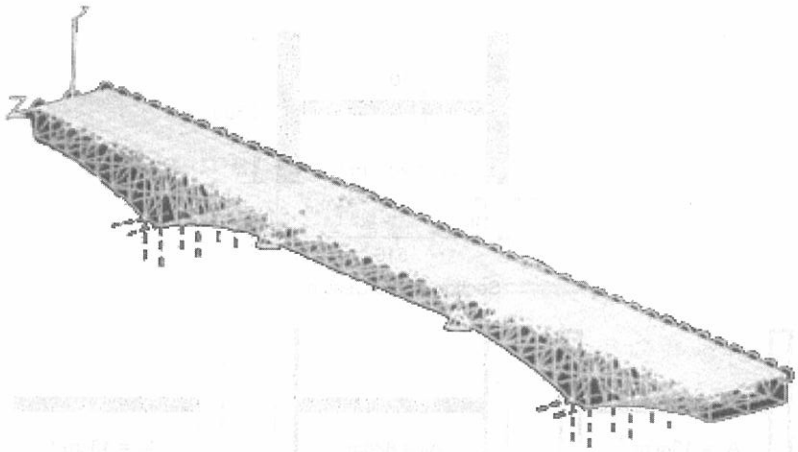


Fig. 4 Numerical modelling of the truss bridge

Table 1. Stresse by various load cases

(σ_{sa} = 1400, σ_y = 2400, σ_f = 4100 : unit kg/cm^2)

Section	Effective section Ae(cm^2)	Dead Load	DL24 (including impact)	DL18 (including impact)	Dead Load +DL24	Dead Load +DL18
Design state	228	729.4	332.2	249.1	1061.6	970.5
Initial condition(1)	138	1205.1	566.2	424.6	1771.3	1629.7
Initial condition(2)	88	1889.8	887.9	665.9	2777.7	2555.7
Initial condition(3)	48	3464.6	1627.8	1220.8	5092.4	4685.4
1st crack progress	85.9	1935.9	909.6	682.2	2845.5	2618.1
Before collapse	78.2	2126.6	999.1	749.4	3125.7	2876.0

Table 2 Average Daily Traffic(ADT) in the bridge

Year	Motor-car	Bus			Truck			Special Truck	Total
		Sum	Small	Large	Sum	Small	Large		
1980	55,524	7,658	4,412	3,246	18,106	11,493	6,613	2,437	83,725
1981	59,254	8,172	4,709	3,463	19,322	12,265	7,057	2,601	89,349
1982	63,235	8,721	5,025	3,696	20,621	13,090	7,531	2,775	95,352
1983	67,484	9,307	5,363	3,944	22,006	13,969	8,037	2,962	101,759
1984	72,018	9,933	5,723	4,210	23,485	14,908	8,577	3,161	108,597
1985	59,775	9,169	5,283	3,886	15,528	9,857	5,671	1,822	86,294
1986	62,777	9,914	6,446	3,468	16,576	11,427	5,149	1,742	91,009
1987	65,779	10,660	7,609	3,051	17,625	12,997	4,628	1,662	95,726
1988	68,781	11,405	8,772	2,633	18,673	14,567	4,106	1,582	100,441
1989	64,752	8,873	7,057	1,816	13,023	10,225	2,798	1,014	87,662
1990	70,565	9,533	7,392	2,143	16,638	11,741	4,897	1,834	98,572
1991	72,784	9,134	6,895	2,239	18,745	15,501	3,244	1,327	101,990
1992	75,680	10,660	8,528	2,132	18,293	13,612	4,681	1,435	106,068
1993	75,937	10,696	8,557	2,139	18,355	13,658	4,697	1,440	106,428
1994	98,025	13,808	11,046	2,762	23,694	17,631	6,063	1,859	137,386

The traffic survey data past 15 years is provided in Table 2.

Since the ADT in Table 2 is based on the two way traffic volume on the bridge, the corresponding ADT of the one way traffic is an half of it. According to Table 2, an average daily traffic of the bridge is approximately 50,000/day considering one way traffic. It should be noted that not all the vehicles but the heavy vehicles such as heavy buses, trucks, and specially equipped vehicles may cause significant stress amplitude variation and may affect the fatigue life in the members of bridge. In the above, the truck traffic causing a significant stress variation to the structure is referred to as the ADTT (average daily truck traffic), and this truck traffic for the bridge being investigated is calculated based on the ADTT. It should be noted that the obtained data may possess the statistical error.

3.2 Standard truck modelling

It becomes necessary to generalize a various shapes of the heavy vehicles into the simplified standard forms of the trucks to effectively compute the stresses occurring in the bridge. The shapes, characteristics, and total weight distribution of major trucks passed the bridge are categorized into two standardized types of truck as a single truck and a semi-trailer truck excluding small cars and buses. The distribution of the total truck weight to each axle and the distance between the truck axles were calculated and shown in Fig. 5.

When we categorize the heavy buses and trucks into the single truck and the specially equipped vehicles into the semi-trailer truck, the total truck traffic of the bridge during the past 15 years constitutes 56,984,702 vehicles, in which 46,295,205 (81.24%) are the single trucks and 10,689,497 (18.76%) are the

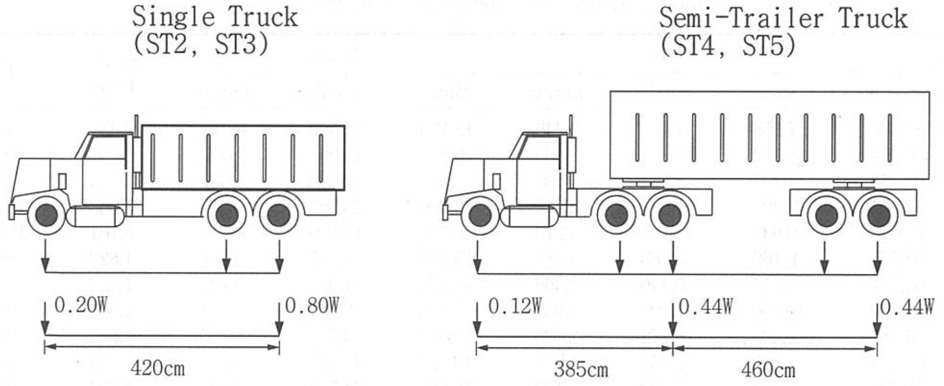


Fig. 5 Two standardized types of trucks

semi-trailer trucks By considering the fact that this bridge had carried the truck traffic which was usually over the design loads, the data in the above is different with those data from 39 sites traffic survey at other roads in the country, which are 90.5% and 9.5%, respectively. This means that the passing ratio of overloaded vehicles was relatively high in this bridge.

4.3 Estimation of fatigue life

The expectancy of fatigue life needs the S-N(stress-number of cycle) curve of the vertical member, which was obtained from experimental test.[3] The Paris curve from the experimental test is $N=4.0*10^{13}*S_r^{-3}$. The weights of all the vehicles passing the bridge are different and the range of stress level is very wide. The Paris curve needs the equivalent stress amplitude range which is obtained from the traffic survey. The procedure to get the equivalent stress amplitude range(S_r) are shown in Table 3. which is 98.23 kg/cm^2.

The fatigue life is calculated by appling the equivalent stress amplitude range, 98.23 kg/cm^2, to the Paris curve. The calculated number of cycles is $42.2*10^6$. According to the design live loads DL-18, the maximum design live load stress including the impact loads is obtained as 258 kg/cm^2. Considering an stress response ratio(α) from load bearing test[5] and vehicle weighting distribution constant(ψ), the equivalent stress amplitude range(S_r) is 90.9 kg/cm^2($\alpha * \psi * S=0.5*0.705*258$). The number of cycles from this value is $53.3*10^6$. The number of cycles from the traffic survey is 26.3% lower than that from analytical method. The differences may originate from the impossible survey on the number of irregularly passing overloaded vehicles and this may be due to the overestimation of the stress response ratio (α), the statistical estimation error on the constant($\psi = \sum \gamma_i \Phi_i^3$) from the truck weight distribution or the other unknown factors. By considering the uncertain factors, the results of the present analysis are acceptable.

Table 3 Equivalent stress amplitude range with only one truck on the bridge

Load Step (Tons)	Mean Volume (Tons)	Axial Force of Vertical p (Tons)	Full Penetration (kg/cm^2)	Partial Penetration S (kg/cm^2)	Volume Ratio r (%)	$r*P^3/100$	$r*S^3/100$
Semi-Trailer Truck							
10 - 12	11	5.98	26.31	46.39	0.40	0.87	403.64
12 - 14	13	7.07	31.10	54.82	0.87	3.09	1439.14
14 - 16	15	8.16	35.88	63.26	2.59	14.06	6550.49
16 - 18	17	9.25	40.67	71.69	1.96	15.48	7211.28
18 - 20	19	10.34	45.45	80.12	1.60	17.68	8237.14
20 - 22	21	11.42	50.24	88.56	2.14	31.83	14829.01
22 - 24	23	12.51	55.02	96.99	1.62	31.68	14759.23
24 - 26	25	13.60	59.81	105.43	2.78	69.98	32600.84
26 - 28	27	14.69	64.59	113.86	2.12	67.14	31278.28
28 - 30	29	15.78	69.38	122.29	1.49	58.43	27218.37
30 - 32	31	16.86	74.16	130.73	2.15	103.18	48063.68
32 - 34	33	17.95	78.94	139.16	1.94	112.30	52312.24
34 - 36	35	19.04	83.73	147.60	1.91	131.75	61371.45
36 - 38	37	20.13	88.51	156.03	1.86	151.69	70661.57
38 - 40	39	21.22	93.30	164.47	1.70	162.19	75555.03
40 - 42	41	22.30	98.08	172.90	0.49	53.84	25081.48
42 - 44	43	23.39	102.87	181.33	0.11	14.49	6751.24
44 - 46	45	24.48	107.65	189.77	0.26	37.97	17686.33
46 - 48	47	25.57	112.44	198.20	0.24	40.55	18891.44
48 - 50	49	26.66	117.22	206.64	0.05	9.19	4281.44
50 - 52	51	27.74	122.01	215.07	0.08	17.27	8045.64
52 - 54	53	28.83	126.79	223.50	0.02	3.88	1805.96
54 - 56	55	29.92	131.57	231.94	0.02	4.33	2018.22
56 - 58	57	31.01	136.36	240.37	0.21	62.69	29204.39
58 - 60	59	32.10	141.14	248.81	0.00	0.00	0.00
60 - 62	61	33.18	145.93	257.24	0.03	11.82	5506.81
62 - 64	63	34.27	150.71	265.67	0.05	19.53	9099.62
		Sum			28.68	1246.93	580863.95
Single Truck							
2 - 4	3	1.79	7.86	13.86			
4 - 6	5	2.98	13.10	23.10			
6 - 8	7	4.17	18.35	32.34			
8 - 10	9	5.36	23.59	41.58			
10 - 12	11	6.56	28.83	50.82	15.62	44.01	20501.74
12 - 14	13	7.75	34.07	60.06	19.00	88.40	41177.41
14 - 16	15	8.94	39.31	69.30	8.42	60.13	28012.51
16 - 18	17	10.13	44.56	78.54	8.49	88.35	41157.24
18 - 20	19	11.32	49.80	87.78	6.01	87.25	40645.17
20 - 22	21	12.52	55.04	97.02	3.93	77.13	35930.54
22 - 24	23	13.71	60.28	106.26	3.82	98.31	45796.03
24 - 26	25	14.90	65.52	115.50	2.84	93.88	43731.84
26 - 28	27	16.09	70.77	124.74	1.90	79.11	36852.99
28 - 30	29	17.28	76.01	133.98	0.98	50.53	23538.23
30 - 32	31	18.48	81.25	143.22	0.25	16.05	7475.46
32 - 34	33	19.67	85.49	152.47	0.06	4.47	2081.00
		Sum			71.32	787.62	366900.16
		Total Sum				2034.55	947764.11
		Sr.eq.				12.67 (Ton)	98.23 (kg/cm^2)

5. Conclusions

Based on the present study, several conclusions are drawn and summarized in the following. (1) It is found that the vertical member, one of the main members, improperly welded at the construction time with the reduced effective section area led to shortening the fatigue life of the bridge. (2) The findings confirm that the fatigue life of the bridge obtained based on the traffic survey is similiar with the real life of the bridge so far. (3) It is clear that the heavy vehicles, which carry the loads over the design load, significantly reduced the bridge life. (4) Although the traffic volume of the overloaded heavy vehicles is small, those effects to the damage of the bridge are significant.

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