Measurement of Service Stress and Fatigue Life Evaluation of Bridges

Mesure des contraintes à l'état de service et évaluation de la durée de vie des ponts

Betriebsspannungsmessungen und Lebensdauerberechnungen von Brücken

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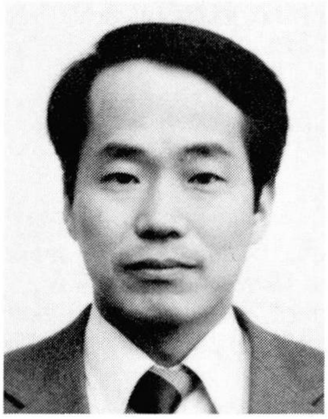


Kentaro Yamada, born 1946, received his civil engineering degree at Nagoya University, and PhD at University of Maryland, MD, USA. Since then, he has been involved in fatigue tests of welded structures, application of fracture mechanics, and field measurement of service stresses of highway bridges.

SUMMARY

Service stresses have been measured by using a histogram recorder. Fatigue life is evaluated using the Miner's cumulative damage rule and the ECCS fatigue-strength diagrams. Calculated fatigue lives of main members are longer than a hundred years whereas for secondary members, relatively short. These results are consistent with the observation of major fatigue cracks in secondary members.

RÉSUMÉ

Les contraintes à l'état de service ont été mesurées à l'aide d'un enregistreur d'histogramme. La durée de vie est évaluée en utilisant la loi de cumul du dommage de Miner et les courbes de résistance à la fatigue du CECM. Les durées de vie calculées pour les éléments principaux sont supérieures à cent ans, alors que celles des éléments secondaires sont relativement courtes. Ces résultats sont en accord avec les observations des principales fissures de fatigue recensées dans les éléments secondaires.

ZUSAMMENFASSUNG

Mit einem Histogramm-Registriergerät wurden Betriebsspannungen gemessen. Für die Berechnung der Lebensdauer werden das Gesetz von Miner über die Schadensakkumulation und die EKS-Ermüdungsfestigkeitskurven verwendet. Die so berechnete Lebenserwartung beträgt für die Haupttragelemente mehr als hundert Jahre, während sie für die untergeordneten Elemente relativ gering ausfällt. Diese Resultate stimmen mit der Beobachtung überein, dass die grössten Ermüdungsrisse in untergeordneten Elementen auftreten.

1. INTRODUCTION

Both stresses and their frequency of in the structural members of highway bridges have increased due to an increasing weight and number of trucks in the past two decades in Japan. It has led to some fatigue cracks in structural members in highway bridges [2,3]. Although the cracks were mainly found in the secondary members and caused no immediate danger to the bridge, they may grow in size with stress cycles and cause serious maintenance problems to the bridges. In order to investigate the cause of cracks, stress measurements were often carried out.

This study was to evaluate the fatigue life of structural members in five steel bridges based on the measured stress range histograms. The bridges include a riveted truss bridge, a composite plate girder bridge, a non-composite plate girder bridge, a box girder bridge with orthotropic steel deck and an arch bridge subjected to different traffic conditions.

2. SERVICE STRESS MEASUREMENT OF BRIDGES

2.1 Description of Bridges

1) Bridge A: It is a 60-year old riveted Warren truss bridge, as shown in Fig. 1, with a span of 73 m and a 16 m wide roadway. The bridge is daily subjected to about 250 trains at a speed of about 10 km/h and about 10,000 vehicles, in which 11 percent are trucks. The traffic is considered rather light in Japan.

2) Bridge B: It is a welded plate girder bridge built in 1967, with a span of 27.9 m and a 7.0 m wide roadway for two lanes, as shown in Fig.2. The spacings of main girders are 2.0 m and the sway bracings are placed at 5.58 m intervals. The reinforced concrete deck was originally 23 cm thick, and was later added by a 12 cm thick steel-fiber reinforced concrete in 1979. It is a private bridge in a steel works. Although about twenty special trucks of more than 785 kN (80 tons) pass on it daily, the number of daily truck traffic is very small.

3) Bridge C: It is a two-span continuous plate girder bridge built in 1964, with eight main girders spaced at 3.2 m intervals, as shown in Fig.3. The total length is 40 m with two spans of 20 m each. The roadway is for six lanes, threc lanes for each direction. The deck is made of 19 cm thick reinforced concrete slab. The bridge situates in an industrial area and carries about 100,000 vehicles daily. About half of the passing vehicles are trucks and it is considered to be under severe traffic condition. The bridge was repaired and retrofitted in 1982, when cracks were found in the fillet weld at the upper end of vertical stiffener. The stress measurement was carried out in 1984 [4].

4) Bridge D: It is a three-span box girder bridge with orthotropic steel decks, built in 1964. It has a 96 m center span and two 77 m side spans, as shown in Fig.4. It is situated on the same route as Bridge C, and is also subjected to severe traffic condition. Several fatigue cracks were found in the corner of diaphragms, and rehabilitation works were carried out in 1989.

5) Bridge E: It is a two-hinge deck type arch bridge built in 1964. The arch has 54 m span and the width of roadway is 9.9 m, as shown in Fig.5. The bridge is subjected to about 80,000 vehicles every day, and 26 percent of them are trucks. The bridge is also subjected to severe traffic condition. Fatigue cracks were found in stringers of floor system and vertical members of arch. The cracks of the stringers were repaired and the cracked parts were stiffened. Further repair and strengthening of the arch and replacement of concrete slab into orthotropic steel deck are underway.

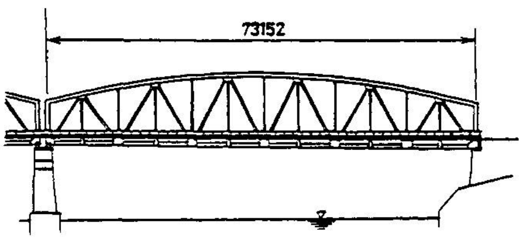


Fig.1 Bridge A: Riveted truss bridge

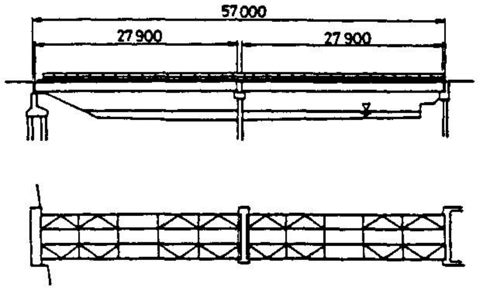


Fig.2 Bridge B: Composite plate girder bridge

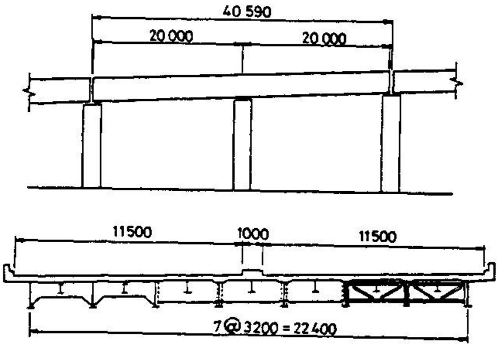


Fig.3 Bridge C: Non-composite plate girder bridge

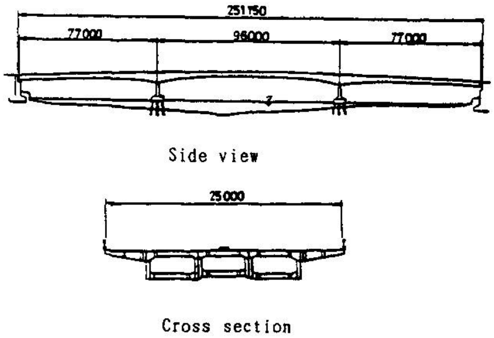


Fig.4 Bridge D: Three span continuous box girder bridge

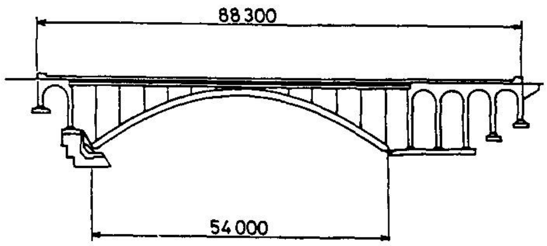


Fig.5 Bridge E: Deck type arch bridge

2.2 Measurement under Model Vehicle of Controlled Weight

Trucks of controlled weight of 196 kN (20 tons) are often chosen to be model vehicles in order to represent the design truck T-20. The model vehicles are placed at several points on the bridge or driven at a certain speed through the bridge. Static and dynamic strains are measured by using strain gages. The dynamic strains are recorded in magnetic tape data recorder, and then monitored by a synchroscope or a pen-recorder. The test results are often compared with the analytical results. This type of measurements causes some difficulties as to stop the normal traffics, which are often very heavy in Japan.

2.3 Measurement of Service Stress Range Histogram

Stresses under service condition are important to evaluate the durability of structures. Service stresses can be measured by using a Histogram Recorder, as shown in Fig.6 [9]. It analyzes the strain waves in real time and records the stress ranges and the number of their occurrence. The analysis may be based on rainflow counting method, peak counting method, level crossing counting method, and so on. The rainflow counting method was used in this study to obtain stress range histogram. The Histogram Recorder has some advantages as that it requires no traffic control, and the disadvantage is that it distinguishes neither the type of trucks nor their axle weight. Therefore, stresses under model vehicle with controlled weight are often measured and used as reference values.

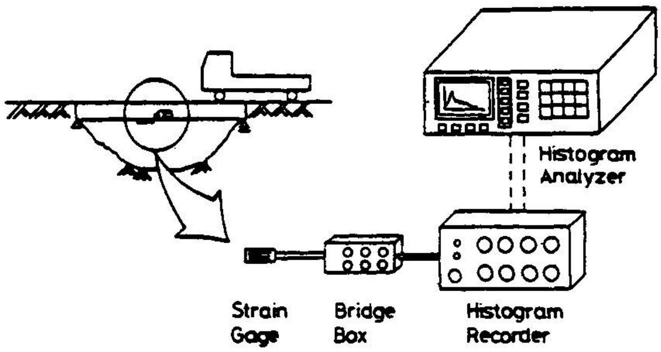


Fig.6 Measurement of stress histogram

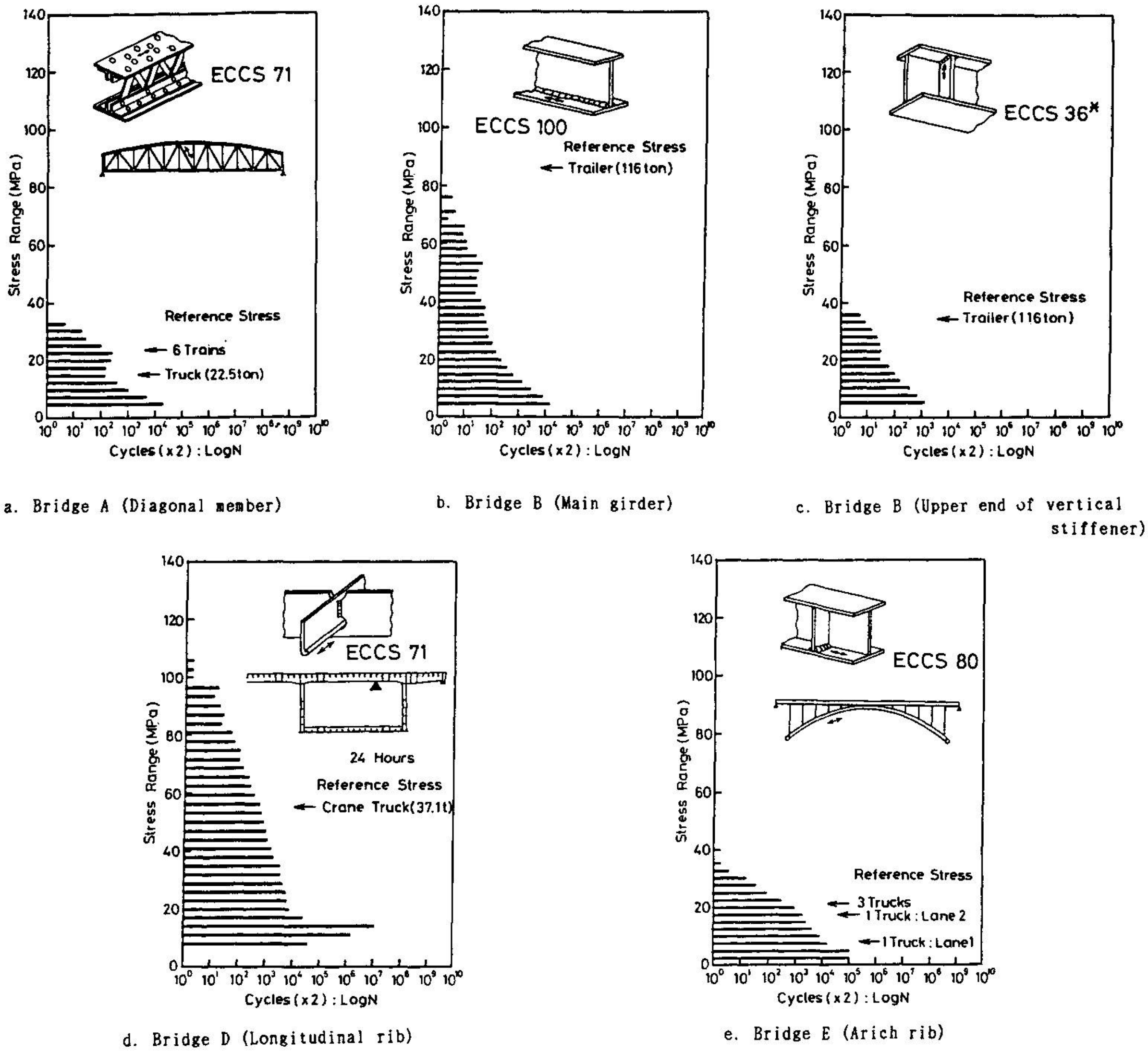
Examples of the stress range histogram measured for 24 hours are shown in Fig. 7, where the ordinate indicates stress range and the abscissa shows the number of half stress cycles in logarithmic scale. The stress ranges corresponding to the model vehicles are also shown as reference values.

The stress histogram of a diagonal member of the Bridge A is shown in Fig.7a. The model vehicles are a 22.5-ton dump truck with 3 axles and a train with 6 cars. It seems that the upper peak of stress cycles is caused by the trains. The stress cycles below the reference value by the model truck may be caused by lighter trucks and the other vehicles and that above the reference value by the train is considered to be caused when the trucks and the train passed on the bridge simultaneously.

Figs.7b and 7c show stress range histograms measured at a lower flange and the upper end of a vertical stiffener of Bridge B, respectively. The measured maximum stress range in the lower flange is close to reference stress range by the 1140 kN (116 tons) trailer truck. And that in the lower flange of Bridge B is about 80 MPa, about twice of that in the diagonal members of Bridge A.

Fig.7d shows stress range histogram measured at bottom of longitudinal rib in the orthotropic steel deck of Bridge D. The model vehicle was a crane truck of 364 kN (37.1 tons). The maximum stress range was about 105 MPa, about twice of stresses due to the model vehicle. It is either because the trucks weigh more than the model vehicle, or because heavy trucks crossed the bridge simultaneously. There was extremely large number of low stress ranges. This may be explained by the frequent passage of small trucks or vibration in the orthotropic steel deck due to the passing traffics.

Fig.7e shows stress range histogram measured at the arch rib of passing lane side of Bridge E. The reference stress ranges due to a truck of 206 kN (21 tons) passing on the slow lane or on the passing lane, or three same trucks are also shown in the figure. The measured stress range were larger than the stresses by the three trucks. It implies that vehicles weighed more than the model trucks frequently passed on the bridge.



a. Bridge A (Diagonal member)

b. Bridge B (Main girder)

c. Bridge B (Upper end of vertical stiffener)

d. Bridge D (Longitudinal rib)

e. Bridge E (Arich rib)

Fig.7 Example of stress range histogram records

The stress range histograms of Bridges D and E were different from that of Bridges A and B. This is because the volume of traffics is much greater in Bridges D and E than in Bridges A and B. For example, about 250 trains and 10,000 vehicles passed on Bridge A, while over 100,000 vehicles passed on Bridge D and about 80,000 vehicles passed on Bridge E for 24 hours.

3. EVALUATION OF FATIGUE LIFE BASED ON MEASURED STRESS RANGE HISTOGRAM

3.1 Detail Categories

In this study the design S-N diagrams specified in the ECCS Recommendation for the Fatigue Design of Steel Structures [8] were used to evaluate the fatigue strength of weld details. Since the design S-N diagrams correspond to the mean-2xs (s:standard deviation) S-N diagrams, the computed fatigue life is the lower bound fatigue life. The design S-N diagrams are given for classified structural detail. The detail categories evaluated for fatigue life in this study are also shown in Fig.7.

1) Bridge A: Riveted joints are classified as ECCS 71.

2) Bridge B and C: The lower flange of the plate girder is fillet-welded to the web, and this detail is classified as ECCS 100. The upper end of the vertical stiffener (connection plate) with a cross beam or a lateral bracing can be considered as load-carrying fillet weld, and it is ECCS 36 if root-cracking is assumed.

3) Bridge D: The stress range of longitudinal ribs welded to lateral ribs in the orthotropic steel deck is used from that measured at the bottom of the longitudinal rib. The welds are assumed as load-carrying fillet weld and it is classified as ECCS 71 for toe crack.

4) Bridge E: The upper and the lower ends of the vertical members are fillet-welded to splice plates, which are then riveted to the arch rib and the floor system. The weld detail is classified as ECCS 71. The arch rib has stiffeners that are fillet-welded to the arch rib web. The detail is non-load-carrying fillet weld, and is classified as ECCS 80.

3.2 Evaluation of Fatigue Life Based on Service Loading

Two types of design S-N diagrams specified in the ECCS Recommendation are used to evaluate the fatigue damage based on the stress range histogram, as shown in Fig.8.

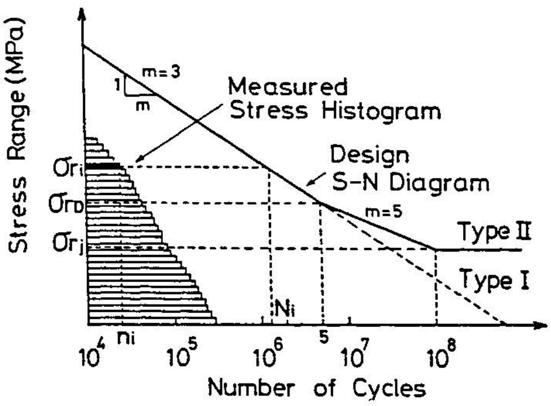


Fig.8 ECCS design S-N diagram

1) Type I (Modified miner's rule): The S-N diagram of slope of m=3 is extended linearly.
2) Type II (Three line method): The slope of the S-N diagram m is equal to 3 for N less than 5 million cycles, and 5 for N between 5 million and 100 million cycles. Stress ranges below the cut-off limit are neglected.

To carry out the damage calculation, the actual number of stress cycles n_i corresponding to each stress range σ_{ri} is obtained first from the histogram. Then the number of cycles to failure N_i corresponding to each σ_{ri} is obtained from the S-N diagram. The fatigue damage D is the sum of n_i/N_i for all stress range in the histogram, i.e.,

$$D = \Sigma\ (n_i/N_i) \qquad (1)$$

where it is assumed that the detail fails due to fatigue, when D becomes unity. Since all the measured stress range histograms in this study are for 24 hours, the evaluated D according to Eq.1 indicates the fatigue damage in one day. Assuming that the data is obtained in a typical day and that the traffic condition will remain unchanged, the expected fatigue life L is given by the following equation.

$$L(\text{year}) = 1/(365 \times D) \qquad (2)$$

The fatigue life may also be evaluated by using the equivalent stress range concept. The equivalent stress range $\sigma_{r,eq}$ is obtained from the S-N diagrams by assuming that the sum of n_i/N_i becomes unity at the time of fatigue failure. When one wants to use the type II S-N diagram (three line method), those stress range below the fatigue limit are considered by using slope m = 5 and those below the cut-off limit are neglected in the calculation of $\sigma_{r,eq}$ [8].

4. FATIGUE EVALUATION AND DISCUSSION

4.1 Influence of Traffic

Fatigue lives of the above-mentioned structural details in the Bridges A and B computed from the measured stress histograms are plotted in Fig.9. The two bridges were subjected to relatively light traffics, and their fatigue life are shown by the solid symbols in Fig.9. The fatigue life of the main members of Bridge A yields to more than 220 years, and that of the tension flanges of Bridge B are between 120 and 140 years. Even the upper end of vertical stiffeners of Bridge B showed the life longer than 90 years. Considering that the evaluated fatigue life is the lower bound because of the use of the lower bound S-N diagrams, it may be safe to concluded that fatigue cracks would hardly occur in these details, provided that traffic condition will remain unchanged and structural deterioration such as corrosion will not take place.

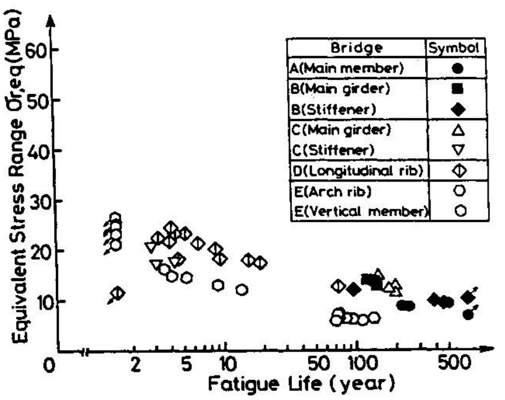


Fig.9 Equivalent stress range and fatigue life for Bridges A and B

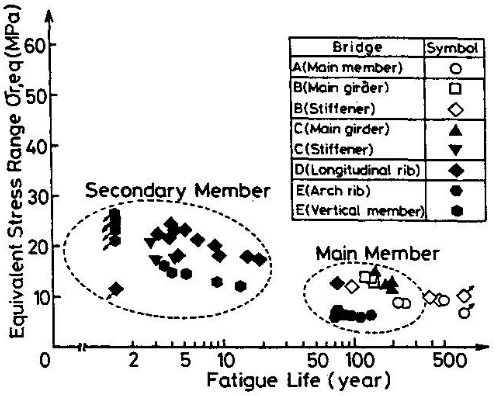


Fig.10 Equivalent stress range and fatigue life for Bridges C, D and E

Bridges C, D and E were subjected to severe traffic condition, and their fatigue lives are plotted by the solid symbols in Fig.10. If the structural details are grouped into main members and secondary members, the computed fatigue lives can be also grouped accordingly. For example, the fatigue lives of the tension flange of Bridge C are more than 100 years, and that of the arch ribs of Bridge E is between 70 and 130 years. On the contrary, the fatigue lives of the vertical members of the arch are less than 4 years, that of the longitudinal ribs in Bridge D are less than 20 years, and that of the upper and lower ends of the vertical members of Bridge E are below 15 years. Actually, the welded details in these secondary members were found susceptible to fatigue cracking previously [2,3,4]. Therefore, the fatigue lives obtained from the measured stress histogram are a good indicator of the severeness of structural detail against fatigue, even though the fatigue lives computed here are still inaccurate due to many uncertainties involved in the analysis.

Bridges B and C are both plate girder bridges with similar structural details, and their fatigue lives are plotted by the solid symbols in Fig. 11. It is noted that the main girder of Bridge B showed shorter fatigue life than Bridge C, although Bridge C are subjected to about 100,000 vehicles daily. This may be because Bridge B carries special vehicles of of over 785 kN (80 tons). However, the upper ends of the vertical stiffeners in Bridge B showed long fatigue lives. This may be due to the 35 cm thick concrete slab and the 2.0 m girder spacing in Bridge B. They reduced the stresses and hence increased the fatigue life. On the contrary the Bridge C has a 19 cm thick concrete slab and 3.2 m girder spacings.

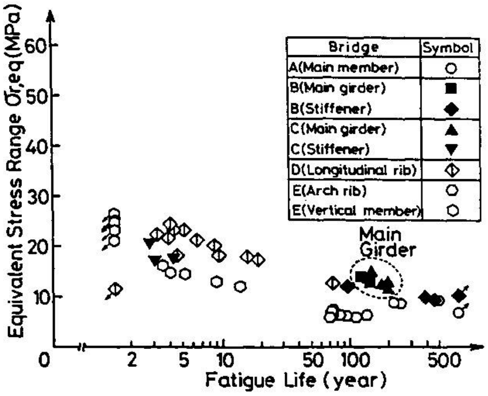


Fig.11 Equivalent stress range and fatigue life for Bridges B and C

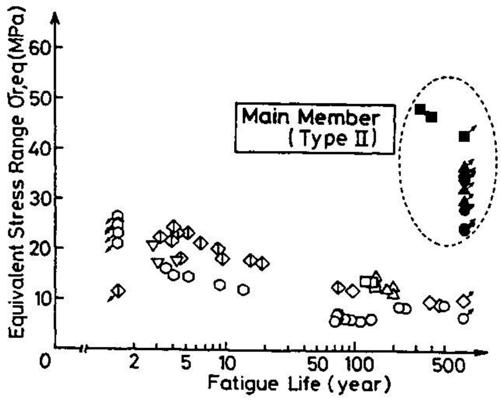


Fig.12 Equivalent stress range and fatigue damage for main members

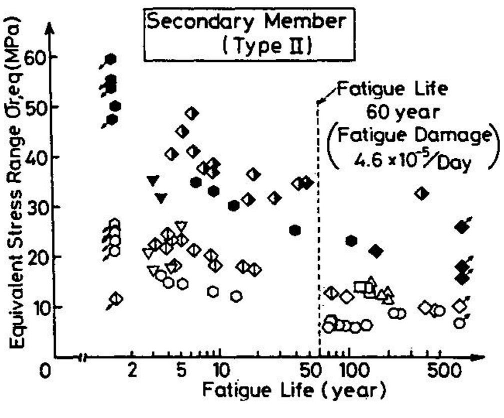


Fig.13 Equivalent stress range and fatigue damage for secondary members

4.2 Influence of S-N Diagrams

The type II S-N diagram (Three line method) was also used to evaluate the fatigue life of each detail, and the results are plotted in Figs. 12 and 13. In Fig. 12, computed fatigue life of the main members (the tension flanges and the arch ribs) using the type II S-N diagram are shown by the solid symbols. They are over 300 years, longer than that evaluated by using the type I S-N diagram. This is because most of the stress ranges in the histograms are below the cut-off limit and they are neglected in the type II S-N diagram. In the case of the arch ribs of the bridge E, the fatigue life becomes infinite because all stress ranges are below the cut-off limit.

The fatigue lives of the secondary members are shown by the solid symbols in Fig.13. Although the equivalent stress ranges increase, the computed fatigue lives are similar to the lives computed based on the type I S-N diagram. A good part of the stress ranges in the histogram were above cut-off limit and contributed to the fatigue damages of the secondary members. Some secondary members actually experienced fatigue crackings, and therefore this trend can also be used to evaluate the severeness of stresses against fatigue.

The fatigue lives computed here should still be considered as relative values. This is because the fatigue strength in the long life region in the S-N diagrams has not yet been clarified due to the lack of experimental data. The fatigue behavior of structural details in long life region, especially under variable amplitude stress cycles, should be further investigated.

4.3 Fatigue Damage Threshold

We may be able to define fatigue damage threshold for bridge members. The fatigue damage threshold is the damage D_{th}, above which the fatigue cracking may occur in a certain period of time. As mentioned above, the fatigue prone details, known from the past experiences, showed relatively large fatigue damage, and hence yielded to short fatigue lives. The dotted line in Fig.13 clearly separate the fatigue-prone secondary members from the main members. It corresponds to fatigue life of 60 years, or daily fatigue damage of $4.6x10^{-6}$.

5. SUMMARY

The measurement of the service stress ranges is important to evaluate the durability of bridges or similar steel structures against fatigue. In this study, stress histograms were collected for five bridges and evaluated. Then the fatigue damages and fatigue lives of the details were computed according to evaluated the design S-N diagrams recommended by the ECCS. The following summarizes the results.

1) The histogram of service stress ranges may be conveniently measured by a Histogram Recorder.

2) The service stress ranges often showed larger stress ranges than the stresses caused by design trucks, such as T-20 and TT-43.

3) Little possibility of fatigue cracking of the main members of highway bridges exists during service period, because the evaluated fatigue lives were in the order of a hundred years or over.

4) The fatigue lives of the secondary members, which experienced fatigue cracking previously, were much shorter than that of the main members.

5) The selection of design S-N diagrams may yield to large discrepancy in the computed fatigue life based on the service stress range histogram. Thus, it is necessary to establish better design S-N diagram in the long life region based on experimental data.

6) The evaluated fatigue life may not be as accurate as expected, but it has relative meaning in expressing the severeness of stresses in the details against fatigue.

7) The daily fatigue damage of $D = 4.6 \times 10^{-6}$ may be used as a fatigue damage threshold value to identify whether the detail is fatigue-prone or not in highway

bridges.

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