Preventive Maintenance Methods for Fatigue Damage of Bridges

Maintenance préventive pour éviter les dommages par fatigue des ponts

Vorbeugende Instandhaltungsmassnahmen bei Ermüdungsschäden von Brücken

Shigeru NAITO
Assistant Chief
Central Japan Railway Co.
Tokyo, Japan

Makoto ABE
Managing Director
BMC Corporation
Chiba, Japan

Ichiro SUGIMOTO
Researcher
Japan Railway Techn. Res. Inst.
Tokyo, Japan

Hidehiko ABE
Professor
Ashikaga Inst. of Technology
Tochigi, Japan

SUMMARY

Fatigue damage is a problem with steel bridges, especially those of the Tokaido bullet train line, where the train traffic is very heavy. This situation requires measures for prevention and prediction of cracks, because they may propagate rapidly. The prediction method used for the bridges of Shinkansen are discussed.

RÉSUMÉ

Les dommages par fatigue sont un problème des ponts métalliques. C'est en particulier le cas des ponts de la ligne du train à grande vitesse Shinkansen dont le trafic ferroviaire est considérable. Cela exige des mesures de prévention et de prévision, car les fissures se propagent rapidement. Le présent document expose la méthode de prévision utilisée pour le réseau ferroviaire japonais.

ZUSAMMENFASSUNG

Ermüdungsschäden in Form von Rissen sind bei Stahlbrücken ein grosses Problem, insbesondere bei denen der Tokaido Shinkansen-Linie, und erfordern daher entsprechende Massnahmen zur Vorbeugung und Voraussage. Beschrieben werden die von den Bahngesellschaften in Japan eingesetzten Vorsorgemassnahmen.

1. INTRODUCTION

Fatigue is likely to become a problem on lines which pass a large number of trains or are submitted to high working stress, since the number of trains and their speeds are expected to rise. Then the traditional corrective method of bridge maintenance after damage has occurred due to fatigue will be inadequate to keep the steel bridge in sound condition and introduction of a preventive one implementing the technology of damage prediction will be mandatory. In this paper, the past practice of fatigue estimation in steel railway bridges is reviewed and referring to the latest progress in the related studies and development of new technologies, a new method of estimating the service life of steel railway bridges is proposed.

2. FATIGUE ANALYSIS AND ITEMS STUDIED

Steel bridges in service should be examined for fatigue according to the following scheme:

2.1 Observation of Fatigue Damage Already in Evidence

(1) Evaluation of soundness concerning a revealed fatigue crack
(2) Investigation into the cause of a revealed fatigue crack(load-bearing capacity of structural member, stress behavior,study of displacement).

2.2 Prediction of Fatigue Damage

(1) Inspection of fatigue limit
(2) Prediction of fatigue crack occurrence and extent
(3) Estimation of residual service life of a cracked structural member.

2.3 Estimation of Service Life (residual life) of a Structure.

Here, the method of predicting the fatigue damages is described.

3. EVALUATIONS TO PREDICT FATIGUE DAMAGE

Meanwhile, fatigue damage may be predicted in terms of "fatigue limit" or "degree of fatigue damage. To see if a fatigue damage is likely to happen, an inspection is made for "fatigue limit". On the other hand, an inspection for "degree of fatigue damage" is done when, during inspection for "fatigue limit", the maximum stress range exceeds the fatigue limit; and it is made for predicting the occurrence timing of fatigue damage. For analysis of the degree of fatigue damage, the method by "cumulative damage" and one by "fracture mechanics" are available.
Only the method by the accumulation fatigue damage is shown. Fig.1 shows assessment of fatigue damage in steel railway bridges.

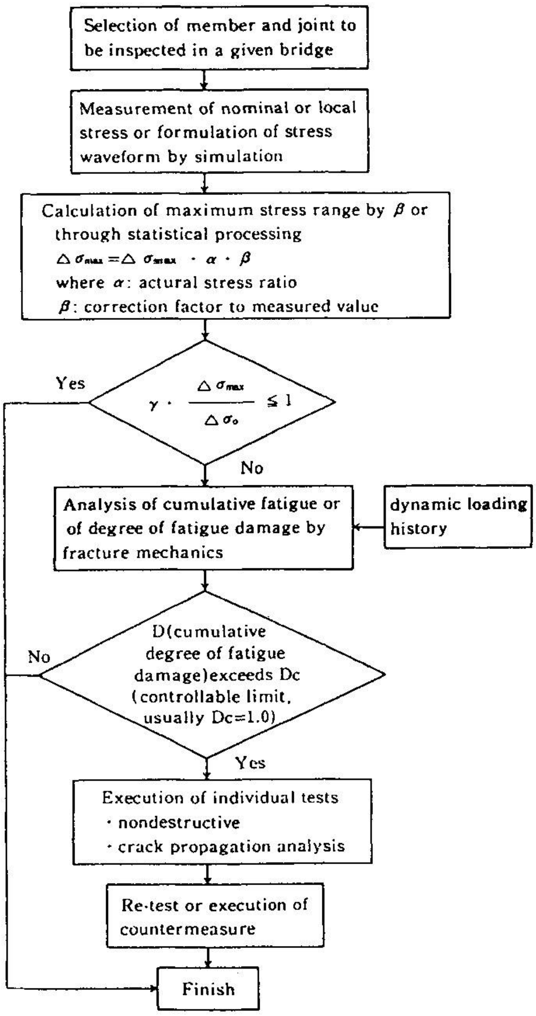

Fig.1 Assessment of fatigue damage

3.1 Assessment for Fatigue Limit

A problem with a structural member as to whether it will raise a fatigue concern in future may be approached from the theory of the so-called "fatigue limit" that the member, such as a joint, will not develop a fatigue failure unless the limit of stress range (fatigue limit) corresponding to a definite vibrational stress of the joint is surpassed, that is, using the following condition:

$$\gamma \cdot \frac{\Delta\sigma_{max}}{\Delta\sigma_0} \leq 1 \qquad (1)$$

with $\Delta\sigma_{max} = \Delta\sigma_{smax} \cdot \alpha \cdot \beta$ α, β : modified factor

where $\Delta\sigma_{smax}$: maximum stress range simulated or measured
$\Delta\sigma_0$: stress range as fatigue limit

The value of $\Delta\sigma_0$ is taken from Table 1 giving the new design standard.[1] The safety factor of a structure is determined in accordance with redundancy, importance and degree of inspection of the structure. In the case of railway bridges, a rather large value is assigned to redundancy and importance, while a rather small value is assigned to degree of inspection, considering that a periodic inspection is mandatory. Thus in the present practice assuming that good balance is assured among these three elements, the safety factor is normally set: γ=1.0.

Rank	Stress range as fatigue limit $\Delta\sigma_0$ kgf/cm² (MPa)
A	1900 (190)
B	1550 (155)
C	1150 (115)
D	840 (84)
E	620 (62)
F	460 (46)
G	320 · (32)

Table 1 Fatigue limit for joints

3.2 Analysis of the Degree of Fatigue Damage

Degree of fatigue damage due to cumulative fatigue means the degree of accumulated fatigue damage, assuming that fatigue damage of a given joint occurs when the degree of fatigue damage accumulates to 1.0 (law of linear cumulative fatigue damage). In the case of railway bridges the degree of fatigue damage is more often expressed in terms of a single train or of accumulated degree of damage to the joint. Then the degree of fatigue damage D after passage of a single train (or a single unit of working load) is given by Equation.

$$D = \sum \frac{N_{eq}}{N_1} = \frac{1}{N_1} \sum \left[\frac{\Delta\sigma_i}{\Delta\sigma_1} \right]^m \cdot n_i \qquad (2)$$

Where
$\Delta\sigma_i$, n_i : respectively stress range generated by imposition of a single unit of working load, and its repetition cycle; $\Delta\sigma_i = \Delta\sigma_{s(i)} \cdot \alpha$
$\Delta\sigma_{s(i)}$: maximum stress range determined by simulation or measurement
$\Delta\sigma_1$, N_1 : standard stress range and its cycle in which a crack initiates respectively.
N_{eq} : equivalent cycle of action by a single unit of working load, as converted to $\Delta\sigma_1$.

3.3 Actual Stress Ratio α

Actual stress ratio is utilized when the stress obtained through design calculation or simulation is to be converted to "working stress" for evaluation of fatigue and it is given as a ratio between measured stress and design-calculated stress. The actual stress ratio α . α sets the following values by the "Maintenance Standards"[2] now.

Influence line length	Actual stress ratio α
≤ 10m	0.65
≥ 10m	0.75

3.4 The Factor β to Correct the Measured Value

When the measured value is employed in the evaluation of fatigue limit, it will not always occur that the maximum value of stress in the range obtained thereby falls into the maximum stress range employed in the evaluation of fatigue damage. The correction factor β is provided to allow for such circumstance. In that case, β should be determined by first measuring the long-term stress frequency and therefrom calculating the probability density function. When the probability density distribution is a normal logarithm one, the non-excess probability value 97.7% is taken as the maximum value to be used in the inspection for fatigue limit and the ratio of this value to the mean (here root cube mean) in the distribution is set as β.
Accordingly, it is assumed that the product of multiplying the mean of measured values sampled by β is the maximum stress range to be employed in the inspection for fatigue limit. Incidentally, in the case of Shinkansen, the value of β is supposed to be in the range of 1.4–1.7.[3]

4. ESTIMATION OF SERVICE LIFE FOR STEEL RAILWAY BRIDGES

4.1 Service Life for Steel Railway Bridge

"Life" which determines the endurance of steel railway bridge is variously defined. In one definition the life of the bridge is exhausted "when a damage occurs to it, which proves economically and physically fatal to maintenance of its structural strength and function;[4),5)] and thereby the period up to this time may be deemed "physical endurance period considering the structural economy (service life)".

4.2 Basic Formula for Evaluation of Fatigue Damage in Calculation of Service Life

Also in calculation of service life, as stated earlier there are two methods of evaluating the fatigue damage: "cumulative fatigue damage" approach and "fracture mechanics" approach.

4.2.1 Evaluation of cumulative fatigue damage

In the application of the above law to the evaluation of an existing structure for its soundness about fatigue, the fatigue damage is divided into D_{pT} so far accumulated and D_{aT} expected to accumulate in future; and when the sum of the two becomes one, it is considered that fatigue failure will take place.

$$D_{pT} + D_{aT} = 1 \qquad (3)$$

For the $\Delta\sigma$– N diagram to be used for evaluation, the Miner law considering no fatigue limit, the modified Miner law considering no fatigue limit or the Haibach method employing a bi-linear strength line diagram for the long-life zone is available, but here "the revised Miner method with cut-off limit" proposed by Miki et al. is adopted.[6] This method is characterized in that the limit value of stress range which does not contribute to fatigue damage even under variable amplitude stress is introduced in the revised Miner law.

4.2.2 Evaluation of Fatigue Life in the Corroded Member

Usually the designing provides for possible corrosion in the structure and, except for some members, the strength inspection pays little attention to the corroded degree. And even in the maintenance control for fatigue life a decrease in the fatigue strength due to corrosion is more often not considered, although the notched section receives attention. The reasons are that the structures are usually placed under adequate paint control; and from fatigue testing to used girders it is confirmed that supposing a fatigue crack has originated from local corrosion at depth, its propagation stops midway and in consequence the ultimate failure takes place at the smallest section.[7] Thus as far as a actual structure is concerned, it seems that in the fatigue life evaluation we need not be concerned about any local roughness.

Therefore, the evaluation is to be made using the $\Delta\sigma$ -N diagram considering the corrosion to a certain extent, though not all is known about the influence of corrosion, so that wrong or dangerous evaluation may not be made. (see Table 2)

kind of stress	kind of joint		strength class	two million cycle strength	gradient	remark (gauge, direction, etc)
nominal stress	flange	base metal corroded	C	125	3	stress inspected on pure section with no-cut off limit
	rivetted joint	slightly corroded	C	125	3	stress inspected on pure section
hot spot	joint with no definition of nominal stress	weld finished or good bead form	D	100	3	stress measured at hot spot and analized
		non-finished	E	80	3	

Table 2 Two million cycle strenth of joints to be employed in fatigue evaluation of girders in service

4.3 Calculation Formulas for Cumulative Fatigue

4.3.1 Cumulative Fatigue D_{pT} so far Suffered and Cumulative Fatigue D_{aT} for Calculation of Residual Life

When, against the degree of fatigue damage expressed in (2), the cycle number of each stress level of K–class so far suffered and the fatigue life (cycle) at stress $\Delta\sigma_i$ are respectively given as n_i and $_i$; and the cycle number causing equivalent damage to the joint of N_0 =2x10^6 cycle fatigue strength $\Delta\sigma_{fo}$ is given as N_{0eq} ,the cumulative damage D_{pT} can be expressed by

$$D_{Tp} = \sum_{i=1}^{k} \frac{n_i}{N_1} = \frac{N_{oeq}}{N_0} \qquad (4)$$

where, since the equivalent cycle number N_{oeq} is

$$N_{oeq} = \sum_{i=1}^{kp} n_i \cdot \left[\frac{\Delta\sigma_i a}{\Delta\sigma_{fo}}\right]^m \qquad (5)$$

D_{pt} may be rewritten as

$$D_{pT} = \frac{1}{N_0} \sum_{i=1}^{kp} n_1 \cdot \left[\frac{\Delta\sigma_i a}{\Delta\sigma_{fo}}\right]^m \qquad (6)$$

D_{at} may also be written similarly as follows:

$$D_{aT} = \frac{1}{N_0} \sum_{i=1}^{ka} n_1 \cdot \left[\frac{\Delta\sigma_i a}{\Delta\sigma_{fo}}\right]^m \qquad (7)$$

where N_0 : 2 x 10^6 cycles
$\Delta\sigma_i$, n_i : i–th stress range and its cycles
$\Delta\sigma_{fo}$: 2x10 cycle strength of the joint
k_p : number of modes of stress range so far suffered
k_a : number of modes of stress range to be suffered thereafter
m : coefficient determining the gradient of the $\Delta\sigma$– N diagram of the joint
a : actual stress ratio.

4.3.2 Calculation of Residual Life

The cumulative fatigue allowable hereafter, that is D_{at} may be derived from (3) as follows:

$$D_{aT} = 1 - D_{pT} \quad (8)$$

Otherwise, D_{aT} can be given using the residual life (Tr) as follows:
D_{aT} = (annual sum of fatigue damages to be suffered hereafter) x Tr (years) Namely,

$$D_{aT} = \frac{1}{N_0} \sum_{i=1}^{ka} \left\{ n_{aeq(i)} \cdot \left[\frac{\Delta\sigma_{amax(i)}\alpha}{\Delta\sigma_{fo}} \right]^m \right\} T_r \qquad (9)$$

$$n_{aeq(i)} = \sum_{i=1}^{nn} \left\{ n_i \cdot \left[\frac{\Delta\sigma_i}{\Delta\sigma_{amax(i)}} \right]^m \right\} N_y \qquad (10)$$

where

$\Delta\sigma_{amax(i)}$: maximum stress range generated by trains to be operated
$n_{aeq(i)}$: annual equivalent cycle of maximum stress range by each train operated
$\Delta\sigma_{(i)}, n_{(i)}$: each stress range derived from frequency analysis of variable stress generated by each running train; and its repetition cycle, respectively.
nn : number of stress range levels for frequency analysis under passage of one train
N_y : number of trains passed in a year; if not known, it is assumed that N=365 x n_{ad}
n_{ad} : number of trains passing in a day

$$T_r = N_0(1 - D_{pT}) / \sum_{i-1}^{ka} \left\{ n_{aeq(i)} \cdot \left[\frac{\Delta\sigma_{amax(i)}\alpha}{\Delta\sigma_{fo}} \right]^m \right\} \qquad (11)$$

4.3.3 Strength Used for Cumulative Fatigue

Girders in service may comprise ones corroded partially, ones additionally treated in the field, and ones with their nominal stress not exactly known on account of complex structure or joint profile. Various surveys are underway about these anomalies the above-given Table 2 is resorted to.

5. CONCLUDING REMARKS

To cope with fatigue damage to steel railway bridges expected to aggravate with operational speed-up and increase in the number of trains operated, the technical background of methods for evaluation of fatigue damage is reviewed. Against the fatigue damage anticipated hereafter, it would be necessary to implement the technology of predicting the occurrence and propagation of cracks, unlike the traditional practice dealing with the defect after it has been discovered. Such technology is applicable for estimating the residual life of the existing steel bridges. It is important hereafter to upgrade the precision of evaluation by building up the data on the working stress and the fatigue strength of joints. It is believed that introduction of this technology will enable efficient maintenance control and reasonable replacement planning of steel railway bridges.

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