Fatigue Design Criteria on Honshu-Shikoku Suspension Bridges

Le critère de la fatigue dans le cas des ponts suspendus entre Honshu et Shikoku

Kriterien für den Dauerfestigkeitsnachweis der Honschu-Schikoku Hängebrücken

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1. INTRODUCTION

In the construction of Honshu-Shikoku bridges, the use of 40 to 80 kg/mm^2 class steels is considered. Since allowable fatigue stresses for high strength steels have not been determined in any standard for structural design in Japan, they are required to be determined newly.

The Honshu-Shikoku bridges are designed as combined highway-railway bridges and so the width of main trusses is wider than usual railway bridges. The cross frames are designed rigidly in consideration of wind resistance. And these facts increase tortional rigidity of the stiffening trusses, and make it necessary to consider the fatigue effect by not only single track loading but also double track loading.

With the above mentioned background, the allowable fatigue stress for 60 to 80 kg/mm^2 class steels was determined and the cumulative effect of actual train load was discussed. This report is the outline of these studies.

2. ALLOWABLE FATIGUE STRESS

2.1. Classification of types of joints

Various types of joints were classified in accordance with Railway Bridge Standard[1] as shown in Table 1, that is, 4 groups (A ~ D) for normal stress and 3 groups for shearing stress (S_1 ~ S_3).

Tab1 Classification of joint

	Class	Types of joint
Compression or Tension	A	1.Base metal 2.Friction grip bolted joint 3.Groove welded joint (finished) 4.Longitudinal welded joint 5.Flange plate with gasset which is cut together with flange plate
	B	6.Base metal with a stiffener (finished) 7.Flange plate with groove welded gusset plate
	C or D	8.Base metal with a stiffener 9.Base metal with a diaphram 10.Non load carrying fillet welded joint 11.Web plate with a gusset plate
Shear	S1	12.Base metal
	S2	13.Longitudinal fillet weld
	S3	14.Load carrying fillet weld

2.2. General expression of allowable stress

Allowable fatigue stress for each group is given by the expression $\sigma_{a0}/(1-\alpha k)$, where σ_{a0} is a basic allowable fatigue stress, k is stress ratio, and α is a parameter which shows the effect of mean stress.

2.3. Effect of mean stress

Among various types of joints, longitudinal butt welded joints are considered to be affected mostly by the mean stress because stress concentration by the shape is considered comparatively small and the residual stress overlaps in the direction of external force. Dots (·) in Fig. 1 show the result of fatigue tests on longitudinal butt welded joints with various mean stress conditions [2] [3] [4] [5] [6]. The inclination of the straight line connecting these points is about −1/6. Rewriting this inclination by the parameter α in 2.2, we obtain α = 0.7 in tension range and α = 1.4 in compression range.

In the other hand, recent test reports on welded joints like fillet welded joints and welded I beam show that test results are affected by mean stress very little, when mean stress is in the tension range [7] [8]. Fig. 2 is fatigue test results on fillet welded joints carried out by us [9]. Here also mean stress in tension range affects the results very little.

Considering all these results, the values of α were determined as follows: α = 1.0, that is, no effect of mean stress when $k \geqslant 0.3$ (tension), α = 0.7 when $-1.0 \leqslant k \leqslant 0.3$ (tension), and α = 1.4 when $-1.0 \leqslant k \leqslant 1.0$ (compression).

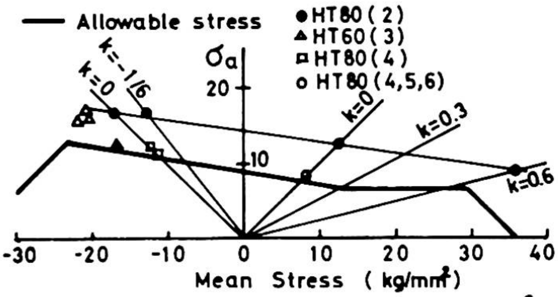

Fig.1 Longitudinal butt joint (2×10^6)

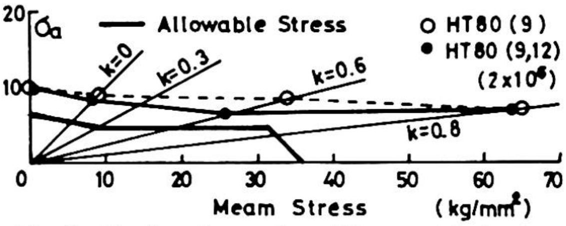

Fig.2 Nonload carying fillet welded joint

2.4. Basic allowable fatigue stress, σ_{ao}

We determined one value σ_{ao} for 60 to 80 kg/mm² class high tensile steel irrespective of the class of steels in convenience of practical usage. The followings are explanations on basic allowable fatigue stress for several typical welded joints.

1) Longitudinal butt welded joint

Fig. 1 shows fatigue strength for 80 kg/mm² class steels obtained from various fatigue tests. As shown in Fig. 1, fatigue strength of 80 kg/mm² class steels is about 15.6 kg/mm² (k = 0) and this value seems rather low. This is a result of approximation of S-N curve by

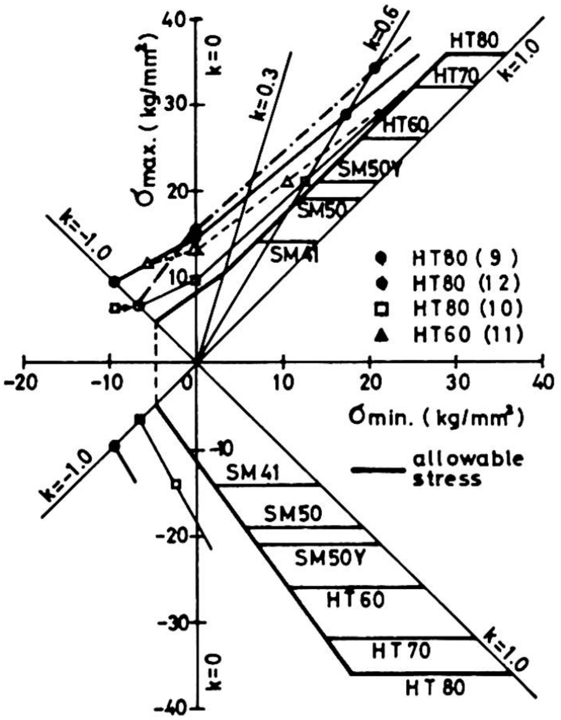

Fig.3 Fatigue strength of fillet welded joint at 2×10^6

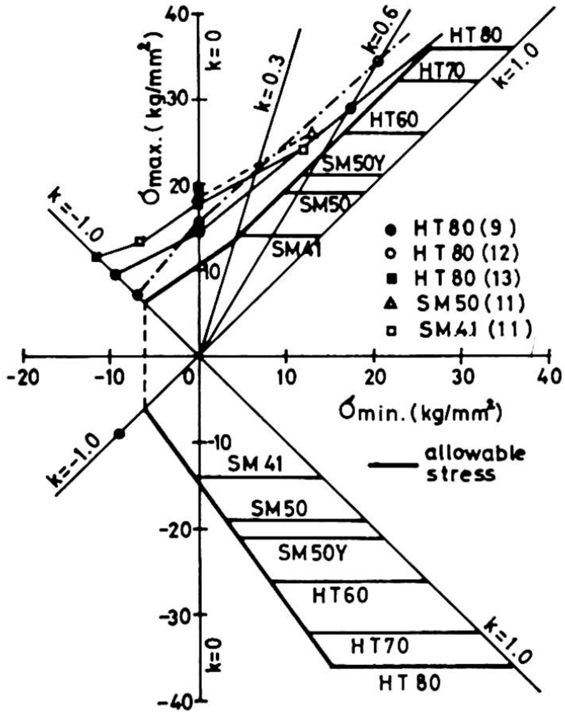

Fig.4 Fatigue strength of fillet welded joint at 2×10^6

a straight line, and there are actually no data which shows fracture by the stress amplitude of less than 16 kg/mm^2. Hence, as for basic allowable fatigue stress for longitudinal butt welded joints, the value σ_{ao} = 1530 kg/cm^2 recommended in Railway Bridge Standard seems appropriate and they are classified in A group.

2) Non-load carrying fillet welded joint

Relatively a large number of fatigue tests on this kind of joints have been carried out. The data of those tests are presented in Fig. 3[10][11]. This figure shows that fatigue strength of higher tensile strength steels tends to be of lower value. Fatigue strength of 80 kg/mm^2 class steels is about 9.8 kg/mm^2. From this result, basic allowable fatigue stress for non-load carrying fillet welded joints must be as low as σ_{ao} = 800 kg/cm^2.

Recently, however, several methods to improve fatigue strength of fillet welded joints have tried[9][12][13]. These method are dressing, finishing, blast treatment and usage of improved welding electrodes which give especially smooth bead. The effects of these methods are shown in Fig. 4. It is clear from Fig. 4 that fatigue strength of improved specimens is increased. So in conditions that the bead and the toe of the weld are well shaped, we can adopt a higher value for the basic allowable fatigue stress. From these results we determined the basic allowable fatigue stress for this type of joint as follows: σ_{ao} = 1050 kg/cm^2 for those in the conditions described above and classified in C group, and σ_{ao} = 800 kg/cm^2 for those without the conditions and classified in D group.

We have been determining basic allowable fatigue stresses for various type of joints listed up in Table 1 with as many test results as possible in the manner explained in this chapter. All the results are presented in Table 2.

Tab.2 Allowable fatigue stress (kg/cm^2)

Class	Tension		Compression
	$-1.0 \le k \le 0.3$	$0.3 \le k \le 1.0$	$-1.0 \le k \le 1.0$
A	$\frac{1530}{1-0.7k}$	$\frac{1355}{1-k}$	$\frac{2160}{1-1.4k}$
B	$\frac{1275}{1-0.7k}$	$\frac{1130}{1-k}$	$\frac{1800}{1-1.4k}$
C	$\frac{1050}{1-0.7k}$	$\frac{930}{1-k}$	$\frac{1480}{1-1.4k}$
D	$\frac{800}{1-0.7k}$	$\frac{710}{1-k}$	$\frac{1130}{1-1.4k}$
	Shearing		
	$-1.0 \le k \le 0.3$	$0.3 \le k \le 1.0$	
S_1	$\frac{920}{1-0.7k}$	$\frac{815}{1-k}$	
S_2	$\frac{820}{1-0.7k}$	$\frac{725}{1-k}$	
S_3	$\frac{650}{1-0.7k}$	$\frac{580}{1-k}$	

(note) $k=|\sigma|_{min.}/|\sigma|_{max.}$ or $|\tau|_{min.}/|\tau|_{max.}$
$k>0$ (in tension or compression only)
$k<0$ (between tension and compression)

3. CUMULATIVE DAMAGE BY TRAIN LOAD

A route (Honshi-Awaji line) of Honshu-Shikoku bridges is planned for double tracks of New trunk lines and D route (Honshi-Bisan line) for double tracks of both New trunk lines and ordinary lines. The total number of trains within 100 years of the service life length is 5.5 x 10^6 in A route and 9.5 x 10^6 in D route. In this chapter, we will discuss cumurative damage by actual load include of the effect of double track loading according to Miner's law based on fatigue strength at 2 x 10^6 loading cycles. Finally we will determine the amendment coefficient for allowable stress.

3.1. S-N curve

S-N curve used in Miner's law is expressed by the following equation and shown in Fig. 5;

$$\log \frac{\sigma}{\sigma_a} = -k_i \log \frac{N}{\bar{N}} \qquad (i = 1, 2) \qquad (1)$$

where σ is stress amplitude, N is loading cycle, σ_a is allowable fatigue strength at 2 x 10^6 and $\bar{N}$ = 2 x 10^6, k_1 = 0.2 ($N \leqslant \bar{N}$), k_2 = 0.1 ($N \geqslant \bar{N}$).

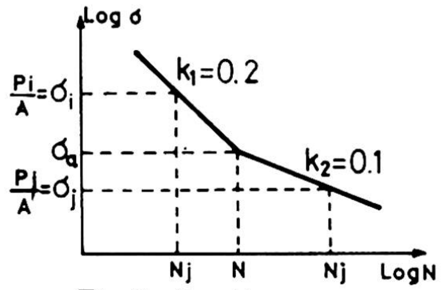

Fig.5 S - N curve

3.2. Cumulative damage by fatigue load[14]

When member force P_i, P_j is given as shown in Fig. 5 that is, n_i cycles of member force P_i where $\sigma_i = P_i/A \geqslant \sigma_a$ and n_j cycles of P_j where $\sigma_j = P_j/A < \sigma_a$, cross sectional area A must be determined so as to satisfy the relation of Miner's law;

$$\Sigma \frac{n_i}{N_i} + \Sigma \frac{n_j}{N_j} \leqslant 1 \qquad (2)$$

Rewriting σ in Eq. (1) by P and A and putting it into Eq. (2), we obtain

$$\frac{1}{(\sigma_a A)^{\frac{1}{k_1}}} \sum_i \frac{n_i}{N} P_i^{\frac{1}{k_1}} + \frac{1}{(\sigma_a A)^{\frac{1}{k_2}}} \sum_j \frac{n_j}{N} \cdot P_j^{\frac{1}{k_2}} \leqslant 1 \qquad (3)$$

P_i and P_j can be expressed as $P_i = f_i P_0$ and $P_j = f_j P_0$, where P_0 is a member force due to the design load. Solving Eq. (3) by A, we obtain

$$A = \beta \frac{P_0}{\sigma_a},\ \beta = \left(\frac{C_1 + \sqrt{C_1^2 + 4C_2^2}}{2}\right)^2,$$

$$C_1 = \sum_i \frac{n_i}{N} f_i^5,\ C_2 = \sum_j \frac{n_j}{N} f_j^{10} \qquad (4)$$

β is an increasing factor of cross sectional area which is obtained by just dividing the member force P_0 by the allowable fatigue stress at 2 x 10^6 loading cycle σ_a. Therefore the amendment coefficient for allowable stress is $1/\beta$.

3.3. Effect of double track loading

When a bridge is loaded by trains on its both tracks at the same time, greater stress fluctuation occurs than single track loading. Here the number of double track loading was calculated from its probability and cumulative damage by both single and double track loading was examined. There are various members where the effect of double track loading must be considered. We will deal with the case of the upper cord member of stiffening truss of suspension bridges as shown in Fig. 6 in this report. Member force fluctuation of a respective member can be obtained from the member force fluctuation line which is drawn by the influence line. Let's assume that a member force fluctuation line is obtained as shown in Fig. 7 from the influence line shown in Fig. 6. First we simplify the line to rectangular waves, and then transform the horizontal axis from distance to time by dividing by the speed of the train as shown in Fig. 8. Fig. 9 is the member force fluctuation diagram of both up and down lines obtained in the same way.

1) Double track loading

Probability of double track loading was calculated only for the cases i) and ii). The cases where $\overline{P}_1^u$ and $\overline{P}_2^d$ or $\overline{P}_2^u$ and $\overline{P}_1^d$ overlap were treated as single track loading because their directions of stress are opposite.

i) Double track loading by $\overline{P}_1^u$ and $\overline{P}_1^d$ (Fig. 10)

The number of double track loading within the service life length can be calculated by the equation

$$N_i = (t_1^u + t_1^d) \times n_t^u \times n_t^d \times 365 \times 100/T.$$

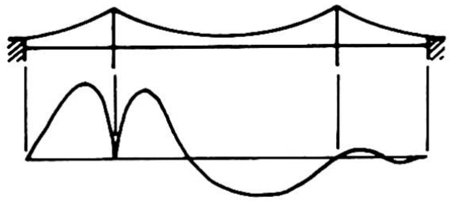

Fig.6 Influence line of upper cord member at the tower link

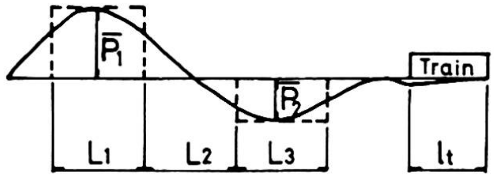

Fig.7 Member force curve

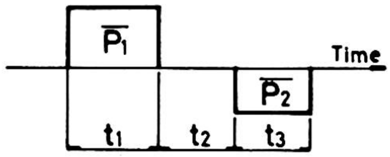

Fig.8 Modeled wave

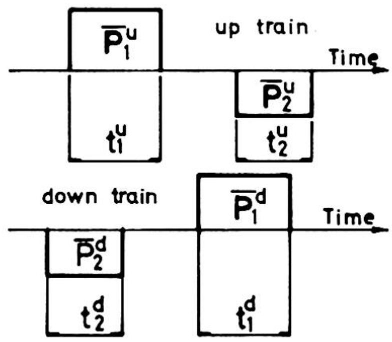

Fig. 9 Single track load

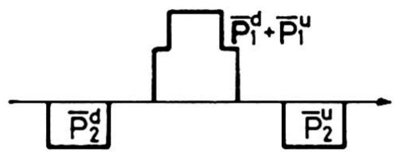

Fig.10 Double track load(i)

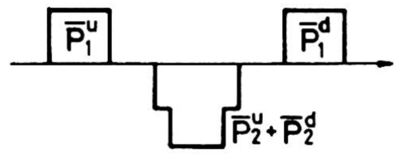

Fig.11 Double track load(ii)

where n_t^u and n_t^d are numbers of trains per day on up and down line respectively and T is service time per day (16 hours). Applying range pair count method to each double track loading, member force fluctuation and number of loading are obtained as follows: (as for the case $\bar{P}_2^d > \bar{P}_2^u$, $\bar{P}_1^d > \bar{P}_1^u$)

a) $P_i^A = \bar{P}_1^d + \bar{P}_1^u + \bar{P}_2^d$, $N_i^A = N_i$, b) $P_i^B = \bar{P}_2^u$, $N_i^B = N_i$.

ii) Double track loading by $\bar{P}_2^u$ and $\bar{P}_2^d$ (Fig. 11)

$$N_{ii} = (t_2^u + t_2^d) \times n_t^u \times n_t^d \times 365 \times 100/T$$

a) $P_{ii}^A = \bar{P}_2^u + \bar{P}_2^d + \bar{P}_1^d$, $N_{ii}^A = N_{ii}$, b) $P_{ii}^B = \bar{P}_1^u$, $N_{ii}^B = N_{ii}$

2) Single track loading

The number of single track loading is obtained by subtracting the number of double track loading from total number of trains.

Up line; $P_s^u = \bar{P}_1^u + \bar{P}_2^u$, $N_s^u = n_t^u \times 365 \times 100 - (N_i + N_{ii})$

Down line; $P_s^d = \bar{P}_1^d + \bar{P}_2^d$, $N_s^d = n_t^d \times 365 \times 100 - (N_i + N_{ii})$

These are calculations for double track lines, however, the same calculation can be applied to four track lines.

3.4. Amendment coefficient for allowable stress

In the checking of fatigue strength, design load for suspended structure is 2.7 t/m per one track and loading length is 370 m in ordinary lines and 400 m in New trunk lines. Service life length is 100 years. The estimated numbers of trains per day on one track in 1990 are as follows,

A route;	New trunk line passenger cars	75 trains
D route;	New trunk line passenger cars	14 trains
	Ordinary line passenger cars	29 trains
	Ordinary line freight cars	86 trains

Based on the above assumptions, the increasing factors of cross section were calculated for suspension bridges in A and D route. We obtained $\beta = 1.23$ for A route and $\beta = 1.30$ for D route. Amendment coefficients were calculated from these values as shown in Table 3. In design of stiffening trusses of suspension bridges, allowable fatigue stress multiplied by amendment coefficients in Table 3 must be used.

Tab.3 Coefficients

Route	Amendment Coefficients
A	0.80
D	0.75

4. CONCLUDING REMARKS

Honshu-Shikoku bridges are expected to be exposed to various conditions which we have never experienced before. And so the fatigue design criteria is rather different from those of usual railway bridges. We doubt that we have obtained enough data to determine the allowable fatigue stress for more than 60 kg/mm^2 class high strength steels and we need more data of various kinds of tests. Now in Fuji city we have a large scale fatigue testing machine which has a capacity of 400 tons dynamic loading. With this machine we are conducting fatigue tests on welded specimens of 80 kg/mm^2 class steels with 75 mm thickness or full size structure models and planning to confirm the allowable fatigue stress and amendment coefficient described in this report.

This report is a summary of studies and discussions by the committee of Honshu-Shikoku bridge super structure of Japan Society of Civil Engineers, sub-committee of fatigue design[10]. The authors wish to express their appreciation to each member of the committee.

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SUMMARY

The fatigue design criteria of Honshu-Shikoku bridges is outlined in two aspects. One is the basic fatigue allowable stress for various types of welded joints of 60 to 80 kg/mm2 class high tensile strength steels and the other is the estimation of the effects of stress and loading cycle by the actual train loads.

RESUME

Le critère de la fatigue dans le cas des ponts suspendus entre Honshu et Shikoku est présenté sous deux aspects. L'un est la contrainte admissible de fatigue principale pour les divers types de joints de soudage de 60 à 80 kg/mm2 pour l'acier à haute résistance à la traction, et l'autre est l'estimation de l'effet de contrainte et du cycle de chargement par la charge actuelle des trains.

ZUSAMMENFASSUNG

Die Kriterien für den Dauerfestigkeitsnachweis der Honschu-Schikoku Hängebrücken bestehen aus zwei Hauptteilen. Der eine Teil ist die zulässige Grundspannung für verschiedene Typen von Schweissverbindungen mit hochfestem Stahl (Bruchfestigkeit 60 bzw. 80 kg/mm2). Der andere Teil ist die Abschätzung des Einflusses der Spannungs- und Belastungswechsel unter der wirklichen Zugbelastung.