Structural Eng. / Earthquake Eng. Vol. 3. No. 2, 297s-307s. October 1986
Japan Society of Civil Engineers (Proc. of JSCE No. 374/I-6)

EVALUATION OF FATIGUE RESISTANCE OF INCLINED HANGERS IN A SUSPENSION BRIDGE BY THE SIMULATION ANALYSIS

By Akio MATSUKAWA, Masahiro KAMEI** and Masao YAMAMOTO****

Fatigue resistance of inclined hangers in a suspension bridge to traffic loads is evaluated. First, it is confirmed that stress ranges and frequencies calculated by the simulation analysis are in good agreement with those measured on a bridge. The stress ranges and frequencies of inclined hangers are estimated by this simulation analysis.

Second, design S-N curves of the parallel wire strand cables are obtained by an approach based on the statistical theory of extremes and fatigue tests. The cumulative damages of inclined hangers are calculated by applying both Miner's rule and the results obtained.

Keywords : fatigue, cable, simulation

1. INTRODUCTION

The significant features of a suspension bridge with inclined hangers are the high structural rigidity and damping due to the truss system composed of a main cable, a stiffening girder and hangers.

However, this truss system causes alternative axial forces on the hanger as the live load passes on the bridge. It also produces larger variable stresses on inclined hangers compared with those of usual suspension structure. In the design of inclined hangers, therefore, the evaluation of fatigue is especially important. Fatigue may be one of the causes of failure on the inclined hangers of Severn Bridge built in 1966[1].

In the study of cable fatigue, it is necessary to clarify the variable stress ranges produced by the traffic load and the fatigue strength of the cables, namely, the S-N curves. It seems that the analytical method of the live load by the computer simulation is effectively applicable to its solution. To date, however, the study has been mainly aimed at the maximum live load[2),3)], and a few have touched on the simulation method of variable live loads[4),5)].

On the other hand, a large number of precious test data on fatigue strength of cables have been presented[6)~9)]. However, there are differences of length, number of cables, etc. between the specimens in the test and actual cables on a bridge. It is important to clarify the effects of these differences on the fatigue strength of the cables, although only a few studies on this subject are available[10)~12)].

In this study, firstly, stress ranges and frequencies produced by the traffic live loads were measured in an existing bridge. Secondly, the reliability of the simulation analysis of the live loads was verified

* Member of JSCE, Dr. Eng., Public Works Bureau, Osaka Municipal Office, (2-2-500, 1-chome Umeda, Kita-ku, Osaka)
** Member of JSCE, Public Works Bureau, Osaka Municipal Office, (2-2-500, 1-chome Umeda, Kita-ku, Osaka)
*** Member of JSCE, Mitsubishi Heavy Industries. LTD. (1-1. 1-chome, Wadasaki-cho, Hyogo-ku, Kobe)

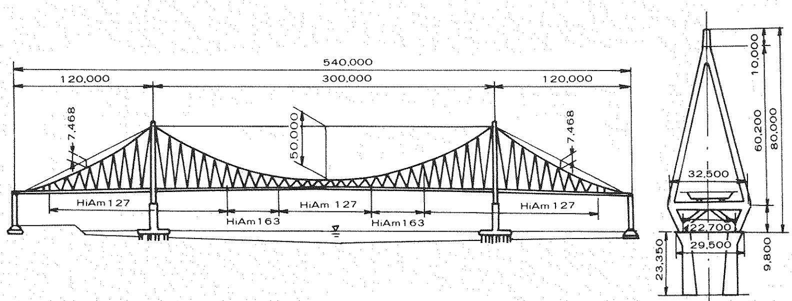

Fig. 1 Bridge model.

comparing it with observed values. Then the effects of the traffic live loads influencing on the distribution of stress ranges and frequencies were investigated.

The design S-N curves in which various influencing factors are taken into account are proposed[13),14)].

The curves are effectively applicable to the fatigue design of cables. Finaly, Miner's rule is applied to cumulative damages in order to evaluate the safety against the fatigue.

2. BRIDGE MODEL AND BASIC DESIGN

(1) Bridge model

The Hokko Bridge investigated in this paper is shown in Fig. 1. The Hokko Bridge is a self-anchored suspension bridge with only one main cable, and the inclined hangers are aligned on the median strip.

The properties of inclined hangers include large compressive stresses and high amplitude variable stresses due to live loading.

Therefore, prestresses are introduced to hangers to protect them against slack, and pararell wire strands with HiAm anchoring system having high fatigue strength are employed.

(2) Basic design of Hokko bridge

In the static design of the inclined hanger, the tensile strength of the hanger wire, safety factor and the allowable tensile stress are determined as 160 kgf/mm^2 or more, 3.0 and 54 kgf/mm^2, respectively.

Considering the secondary stress, the safety factor and the allowable tensile stress are designed 2.0 and 80 kgf/mm^2.

In the fatigue design, 50% of the design live load is rated as the fatigue design load referring to DIN 1073[15)]. According to various tests carried out in Japan and other countries, it is assumed that approximately 25 kgf/mm^2 fatigue strength of HiAm cable is guaranteed. However, in consideration of safety, 20 kgf/mm^2 of fatigue strength is applied. On the basis of the above figures, two types of hangers are adopted : HiAm 127 and HiAm 163 with 7.0 mm wire diameter.

3. SIMULATION OF STRESS RANGES AND FREQUENCIES

(1) Simulation procedure

The stress response of a bridge to the live load greatly depends on the traffic : smooth or jammed.

In this paper, the simulation analysis is made based on the ordinary smooth traffic flow, and a traffic jam. The traffic jam is considered to be a low speed traffic flow.

The deterministic variables of parameters in the analysis are 1) influence values of the member, 2) traffic volume in unit time (Q), 3) density ratio of each vehicle type, and 4) number of car lanes.

The probabilistic variables are 1) vehicle speed, 2) vehicle weight, and 3) arriving time intervals.

Fig. 2 shows the simulation procedures taken in this paper[16)]. First, arriving vehicles are classified into three categories by the uniform random number generated by a computer. Then a line of vehicles is made based on the computer-made random numbers according to the distributed function for weight, speed, and arrival time of each vehicle. Then let this vehicle line pass on the bridge. The generated stress on the

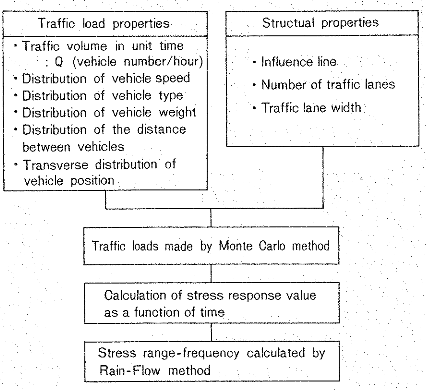

Fig. 2 Simulation analysis procedures.

member here is calculated at a certain time interval. The Rain-Flow method is applied in order to obtain the stress range-frequency relationships from the stress fluctuations.

(2) Definition of parameters

i) Classification of Vehicles : The vehicles constituting the traffic flow are classified into three categories : ordinary car, light truck, and heavy truck.

The traffic density ratio of each vehicle is determined according to the traffic plan in the case of a planned bridge or to the results of actual investigation in the case of an existing one.

ii) Modeling of Vehicle : Fig. 3 shows the model vehicles that represent above categorized vehicles.

iii) Distribution of Vehicle Weight : The weight distribution of vehicles are assumed to be the normal distribution based on the data values in Reference 2).

iv) Distribution of Vehicle Speed : It seems difficult to determine the distribution of vehicle speed because of few statistical data on speed. In this paper, however, the normal distribution is adopted referring to the results of past studies. The measured speed on the actual bridge or the design speed is considered to be the average speed in the normal distribution.

v) Arrival Time Interval : In the smooth traffic, the vehicles shall arrive at random time interval. The distribution of the number of arriving vehicles follows the Poisson distribution. The probability density function of the arriving time interval T_1 is, therefore, expressed by :

$$f_{T1}(t)=\nu e^{-\nu t} \qquad (1)$$

where, ν : is average occurance $(=Q)$.

vi) Traffic Volume : The observed traffic volume on the existing bridge is adopted as the traffic volume, while on the planned bridge the planned traffic volume is employed.

vii) Minimum Headway of Vehicles : The minimum headway of an ordinary car $y_{\min}$ (m)[17] is given by :

$$y_{\min}=5.7+0.14\,V+0.0022\,V^2 \qquad (2)$$

where, V is vehicle speed (km/h). For other model vehicles, the minimum headway values shall be modified in consideration of the vehicle length as shown in Fig. 3.

4. INVESTIGATION OF LIVE LOADS ON AN EXISTING BRIDGE AND CALIBRATION

(1) Investigation of the live load on an existing bridge (Nanko Ohashi Bridge)

The investigation was carried out in the northward lane of Nanko Ohashi Bridge, the three-span Gerber bridge, which connects the reclaimed land in Nanko to a town area. Fig. 4 shows the general view of Nanko Ohashi Bridge. The investigation includes not only traffic volume and the stress ranges and frequencies of actual live loads but also the static and dynamic loading test by test vehicles[18].

a) Static and dynamic load test

The influence line is obtained by loading the test vehicles of which weight is known, with strain gauges attached to the lower flange in the center of the suspended girder. In order to know the impact effects, the dynamic strain values are measured by the running tests of vehicles at various speeds.

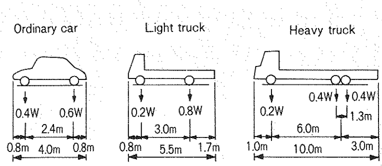

Fig. 3 Vehicle types.

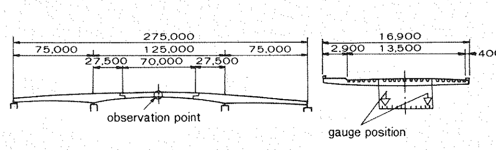

Fig. 4 General view of Nanko Ohashi Bridge.

The measured values obtained by the static load tests are in quite a good agreement, approximate 95%, with the analyzed values. The increasing ratio of the main girder stress due to the dynamic load against the static one is only 5%, which is kept almost constant without depending on vehicle speed. Therefore, dynamic effects are not taken into account in this study.

b) Traffic volume investigation

Table 1 shows the result of the traffic volume investigation.

c) Stress range-frequency measurement

The stress ranges and frequencies are measured by a histogram recorder with strain gages placed on the lower flange at the center of the suspended girder.

The measurement is proceeded for about a month.

Fig. 5 shows the obtained stress ranges and frequencies at different timings in which the frequency is expressed in logarithm. The stress range can be separated into two regions : high stress range and low stress range. Two straight lines can be assumed here : one is the inclination ($\sigma_r/\log N$) of the high stress range as large as 1.9, and the other is 0.51 of the low stress range. It should be noted that the inclination of the stress range-frequency relationships becomes stable if the data is taken for a week or more and can be shifted toward the time direction in parallel to the basic data line. The fatigue analysis in this paper is proceeded on the assumption that the traffic load, obtained by the simulation, passes on the bridge all the time for its life.

Table 1 Vehicle type classification and traffic density of each vehicle.

Classification \ Item	Vehicle type	Number of vehicles	Density ratio
Ordinary car	Private car, light van, microbus	4,863	53%
Light truck	Light truck, small truck	1,033	11%
Heavy truck	Middle sized truck, large truck, bus, tank truck, concrete mixer-truck, trailer, crane truck	3,313	36%

measuring period : AM7:00～PM5:00

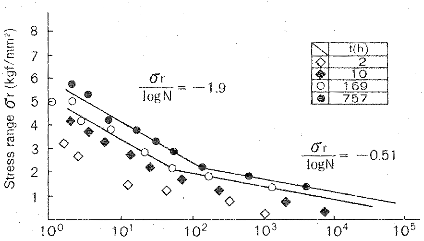

Fig. 5 Stress range-frequency at the lower flange of main girder.

Table 2 Parameters of simulation analysis (Nanko Ohashi Br.).

Items in traffic load condition	Fundamental	Parameters
Traffic volume : number of vehicles/(traffic lanes · day)	4,000	6,000
Average speed : $\overline{V}$km/h	40 (Design speed)	60
Coefficient of variation of vehicle speed : %	10	10
Traffic density ratio of heavy truck : α%	36	54
Average weight of heavy truck : $\overline{W}$ ton	10.5	15.75, 21.0
Standard deviation of heavy truck weight distribution : $\sigma_{\overline{w}}$ ton	7.1	7.1
Times of traffic jam : hour	4	2～4

Average weight of light truck : 3.1 ton
Average weight of ordinary car : 1.2 ton

(2) Results and considerations of the simulation analysis

a) Parameters

The basic parameters in the analysis are shown in Table 2. The traffic volume, the average speed, and the density ratio of heavy trucks in traffic are determined based on the results of field measurments on Nanko Ohashi Bridge. In the case of smooth traffic, the design speed is employed as an average speed of vehicles, while in the traffic jam it is fixed to 2 km/h.

The coefficient of variation of vehicle speed is assumed to be 10% in both cases.

According to the observation at the site, the total period of the traffic jam seems to be about 2～4 hours a day. It is estimated that 4 hours in daytime is a traffic jam in this analysis.

In the parameter analysis, the values of 1.5 times as large as the basic values are used for the traffic volume, the average speed, and the traffic density ratio of heavy trucks as shown in Table 2.

The average weight of heavy trucks is calculated in two ways ; 1.5 times and 2.0 times as large as the basic value. The standard deviations for each avarage weight are kept constant regardless of the average weight, and the basic values are used for them.

b) Results and Considerations of Parameter Analysis

i) Influence of Heavy Trucks

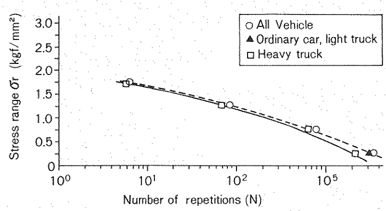

Fig. 6 Stress range-frequency calculated with each type of vehicle.

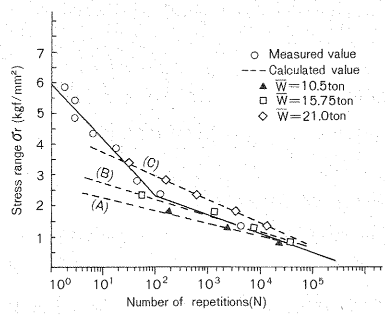

Fig. 7 Influence of average weight of heavy trucks on the condition of smooth traffic.

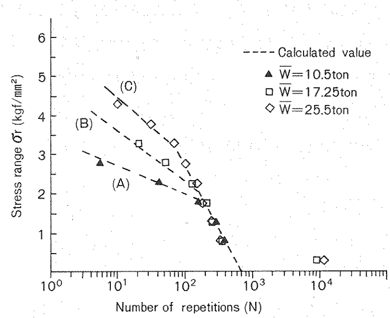

Fig. 8 Influence of average weight of heavy trucks on the condition of traffic jam (for one month).

In order to examine the influence of heavy trucks on the stressrange-frequency relationships in the smooth traffic condition, the additional simulation, that is equivalent to one day actual traffic, is done by giving only heavy trucks' weight without giving any weight of ordinary cars and light trucks. Obviously, as shown in Fig. 6, heavy trucks are the decisive factor in the stress range-frequency relationships. Therefore, heavy trucks should be modeled very carefully. Fig. 7 shows the result of the analysis for one month, in the basic values and in the condition that the average weight are multiplied by 1.5 and 2.0, respectively. The measured values are also indicated in Fig. 7. As shown in Fig. 7, the stress range increases as the average weight of heavy trucks increases. This result suggests that the higher the probability of occurrence of the heavy trucks is, the higher the stress range becomes.

The inclination ratio of each line obtained in three condition is 1.7 : 1.4 : 1.0, which is in good agreement with the average weight ratio of 2.0 : 1.5 : 1.0 at high stress range region ($\sigma_r >$ 1.0 kgf/mm²).

The inclination of stress range-frequency relationships seems to be, therefore, in linear relation to the average weight.

ii) Influence of a Traffic Jam

In the traffic jam model, the simulation is also done by changing the average weight of heavy trucks in three classes as in the smooth traffic condition. Fig. 8 shows the result of the simulation analysis of 120 hours on the assumption of four hour traffic jam per day. As is noted in Fig. 8, the larger the average weight of heavy trucks is, the larger the stress range becomes and its inclination ratio nearly accords with the average weight ratio.

iii) Influence of Traffic Volume and Density of Heavy Trucks

Fig. 9 shows three conditions : 1) basic condition, 2) the condition that is the same as the basic one but its traffic volume is multiplied by 1.5, and 3) the condition same as the basic one but its density of heavy trucks is multiplied by 1.5. In the large stress range region (σ_r=0.5~2.0 kgf/mm²), the repetitions of 2) and 3) above are 1.25 to 2.0 times as large as the bacic condition. The stress ranges and frequencies of both multiplied cases, 2) and 3) coincide with each other.

The reason for this coincidence is that the stress ranges and frequencies depend mostly on heavy trucks. If the number of arriving heavy trucks is equal in both 2) and 3), same stress ranges and frequencies are obtained.

(3) Calibration of analysis values

If each parameter used in the simulation reflects accurately on the actual traffic flow, the stress ranges and frequencies should coincide with the measured values. The following sentenses explain the calibration results of analysis values.

As shown in Figs. 7 and 8, the following may safely be said;

i) The inclination of the measured stress range-frequency relationships on Nanko Ohashi Bridge lies between the lines (B) and (C), and the average weight of the measured values is calculated linearly as $\overline{W}$

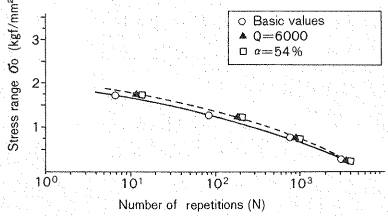

Fig. 9 Influence of number of arriving heavy trucks.

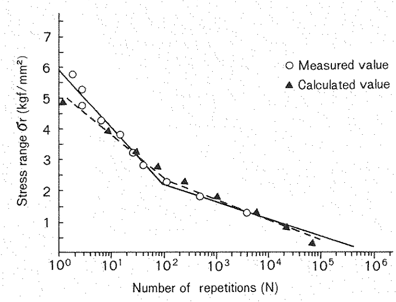

Fig. 10 Simulation analysis result of Nanko Ohashi Br.

=17 t.

ii) Analysis results (A), (B) & (C) in the smooth traffic condition do not show the stress range of more than 3.0 kgf/mm^2. The stress range of more than 3.0 kgf/mm^2 is considered to be produced in a traffic jam.

From the results of the parameter analysis it is clear that the small stress range shall be produced by smooth traffic flow, while the high stress range is by a traffic jam. Under the basic conditions as indicated in Table 2, a simulation analysis is carried out to obtain the time of a traffic jam by setting the average weight of heavy trucks to 17 t, and by different combinations of smooth traffic period ($\overline{V}$=40 km/h) and that of a traffic jam ($\overline{V}$=2 km/h).

The analysis results obtained in the case when the time of a traffic jam is set to 2.5 hours a day show a good agreement with the measured stress ranges and frequencies. Fig. 10 shows the measured stress ranges and frequencies and the analysis result based on the conditions in the above calibration for one month. As shown in Fig. 10, in the region of the large stress range with the frequency of less than 10 cycles, the measured values exceed the simulation analysis result to some extent. It is thought that for the calibration of these values a group of extra heavy trucks has to be taken into account in the analysis.

It is supposed that the stress ranges and frequencies producued by actual live loads can be rather accurately reproduced in this simulation analysis.

5. FATIGUE EVALUATION OF INCLINED HANGERS

The cumulative fatigue damages on the inclined hangers of Hokko Bridge are estimated based on the Miner's rule for the variable live load effect calculated by the simulation analysis and the S-N curve of HiAm cable, which is explained in the following section.

(1) Design S-N curve of HiAm cable

a) Evaluation of Each Affecting Factor

The wire length, number of wire, wire diameter, and the minimum stress are selected as the factors largely affecting on fatigue strength of paralle wire cables, and the fatigue tests (A~D series) of cable wires are carried out to clarify these influences. Table 3 shows the test conditions. Based on the A series, a basic test of cable wires, B, C, and D series are set to evaluate the influences of the minimum stress, the wire diameter, and the wire length.

In Table 3 three kinds of figures are also indicated ; 1) $\sigma_{\min}$: minimum stress applied in this test, 2) σ_r : stress range applied, and 3) σ_0 : standard stress range which is converted from σ_r by the Eq. (3) below.

$$\sigma_0=\{250/(250-\sigma_{\min})\}\cdot\sigma_r \quad (3)$$

The results of fatigue tests carried out under different settings of minimum stress ($\sigma_{\min}$) can be indicated in the same fatigue properties if they are converted into the standard stress range (σ_0). Fig. 11 shows S-N curves with 5% and 50% of the probabilities of failure in A and D series obtained from the results of Weibull analysis. As shown in Fig. 11, the longer the wire length becomes, the lower is the fatigue strength.

b) Evaluation of Length Effects by the Statistics of Extremes

Assuming that the fatigue strength (x) of cable wire of standard length (L_0) follows Weibull distribution, the probability density function ($f(x)$) is expressed by ;

$$f(x)=m\cdot\frac{(x-c)^{m-1}}{\eta^m}\cdot\exp\left\{-\left(\frac{x-c}{\eta}\right)^m\right\} \quad (4)$$

where, m, η and c are shape parameter, scale parameter, and position parameter, respectively.

And the cumulative distribution function ($F(x)$) is expressed by ;

$$F(x)=1-\exp\left\{-\left(\frac{x-c}{\eta}\right)^m\right\} \quad (5)$$

According to the statistical theory of extremes[19], the fatigue strength (x) with the length L ($=n\cdot L_0$, n : multiple of the length effect) becomes the probability density function of minimum value ${}_L\phi_n(x)$ as explained below.

$${}_L\phi_n(x)=n\cdot\{1-F(x)\}^{n-1}\cdot f(x) \quad (6)$$

Substituting Eqs. (4) and (5) into Eq. (6), following is obtained.

$${}_L\phi_n(x)=\frac{m}{\xi^m}(x-c)^{m-1}\cdot\exp\left\{-\left(\frac{x-c}{\xi}\right)^m\right\} \quad (7)$$

where, $\xi=n^{-1/m}\cdot\eta$

When the fatigue strength of a cable wire of standard length (L_0) follows Weibull distribution of three parameters, the distribution of minimum values, according to the Eq. (7), also follows Weibull distribution.

In Eq. (7) shape and position parameters (m & c) are equal to those of Eq. (4) for the standard length (L_0), and only the scale parameters are different.

The shape and scale parameters are obtained from the results of the fatigue test. $C=30$ kgf/mm^2 is used for the position parameter as the results of various studies. Fig. 12 shows the fatigue strength of cable wires (ϕ 7 mm) of three different length ($n=1$, 10^2 & 10^6) that are calculated based on the Eq. (7). The calculations are made on the condition of $L_0=0.2$ m. Fig. 12 also shows the test data.

c) Proposition on Design S-N Curve[14]

The follwing points have been considered to propose the design S-N curve for the fatigue design of cables :

i) It is assumed that even the low level stress range below the fatigue limit works to develop a crack larger as long as higher level of the stress range than the limit exists.

ii) The total fatigue strength of cables in an existing bridge is influenced by the fretting due to the

Table 3 Conditions of fatigue tests of cable wires.

Conditions of tests	Length (mm)	Diameter (mm)	σmin (kgf/mm^2)	σr (kgf/mm^2)	σo (kgf/mm^2)	Number of specimens
A series	200	5.12	10	47.5	49.5	20
				50	52.1	30
				55	57.3	30
				60	62.5	30
				65	67.7	30
				70	72.9	30
B series	200	5.12	25	45	50.0	20
				50	55.6	20
				55	61.1	20
				60	66.7	20
				65	72.2	20
C series	200	7.0	10	45	46.9	20
				50	52.1	20
				55	57.3	20
				60	62.5	20
D series	1000	5.12	10	45	46.9	20
				50	52.1	20
				55	57.3	20
				60	62.5	20
				65	67.7	20

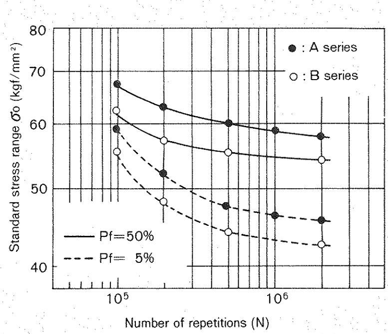

Fig. 11 Effect of wire length.

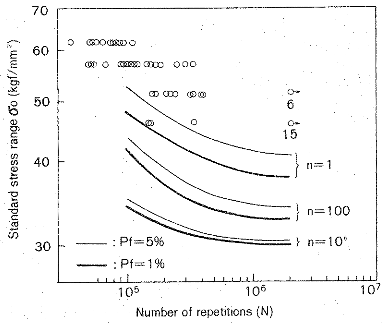

Fig. 12 P-S-N curve considering with the length effect (ϕ 7.0).

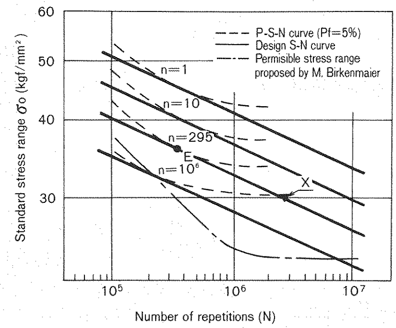

Fig. 13 Design S-N curve (in the case of wire diameter ϕ 7.0).

friction between wires and by additional bending stresses near the socket.

Considering these effects, it is conservatively recommended to use the design S-N curve rather than the P-S-N curve obtained in the preceding section. The test specimen (X : ϕ 7.0 HiAm 37, L=1.6 m) alone showed the probability of failure of 5% at the repetitions exceeding 2×10^6 cycles in the past data.

The multiple (n) against the standard length (L_0=0.2 m) of specimen (X) is $M\cdot L/0.2=295$ when M and L represent the number of wires and the cable length, respectively. As Fig. 13 shows, the experimental data point of the specimen (X) and the tangent point (E) of P-S-N curve of n=295 and the 5% probability of failure is connected by a straight line. The straight line ($\overline{EX}$) is called the design S-N curve of the 5% probability of failure with n=295 for the cable with ϕ 7.0 mm wires.

The design S-N curve for different multiple (n) is going to be obtained by drawing the straight line in parallel with the above straight line ($\overline{EX}$) having contact with the corresponding P-S-N curve.

Thus, the recommended design S-N curve (5 % probability of failure) is expressed by Eqs. (8) and (9) in the case of ϕ 7.0 mm wires.

$$N\sigma_0^{11.8}=C_1 \qquad (8)$$

$$\log C_1=0.0444\{\log(n)\}^2-0.60\{\log(n)\}+25.1 \qquad (9)$$

(2) Stress response of inclined hangers

a) Basis of Simulation Analysis

Fig. 14 shows the influence lines of five significant hangers analized to represent 94 hangers of Hokko Bridge. The simulation analysis is carried out on these five hangers with different influence lines.

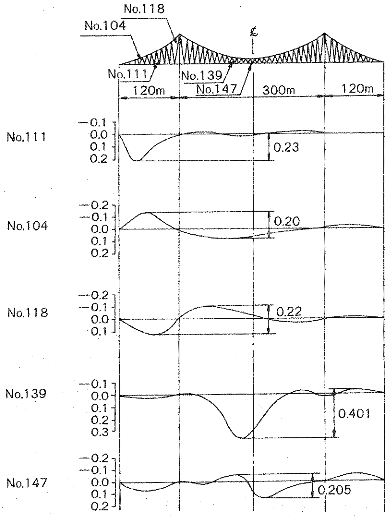

Fig. 14 Influence line of each hanger.

The basic parameters in this analysis are set as follows : The traffic volume per lane and per day is assumed to be 10 thousand vehicles, the planned traffic volume for Hokko Bridge. The average speed is set to 60 km/h, the designed speed, and its coefficient of variation is 10%.

The traffic density ratio of heavy trucks is assumed to be 40% and the average weight of heavy trucks is $\overline{W}$=17 t referring to the result of the investigation of Nanko Ohashi Bridge. The period of the traffic jam is assumed to be 2.5 hours a day. Due to the consideration of the effect of weight of heavy trucks on the fatigue damage, additional simulations are carried out on the average weight of heavy trucks, 1.25 times and 1.5 times as heavy as the basic one.

b) Stress Range-Frequency Relationships for Fatigue Evaluation

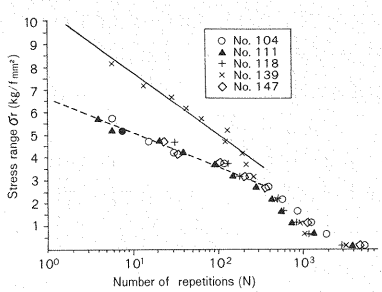

Fig. 15 Stress range-fuequency of each hanger.

Table 4 Cumulative damage of hangers.

Traffic load conditions \ Type of S-N curves \ Hanger no.		No.118	No.139
Fundamental conditions	①	0.166×10^{-1}	0.610×10^{-1}
	②	0.572×10^{-2}	0.395×10^{-1}
Average weight of heavy trucks $\overline{W}=21.25$ton	①	0.217×10^{-1}	0.803×10^{-1}
	②	0.737×10^{-2}	0.453×10^{-1}
Average weight of heavy trucks $\overline{W}=25.5$ton	①	0.570×10^{-1}	0.189×10^{-1}
	②	0.190×10^{-1}	0.989×10^{-1}

Note ① S-N curve by Birkenmaier
② S-N curve proposed by authors

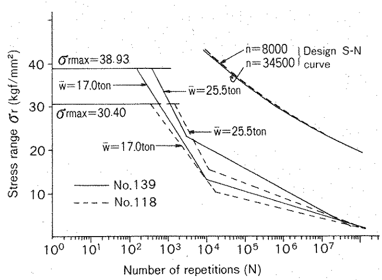

Fig. 16 Stress range-frequency and design S-N curve.

Fig. 15 shows the stress range-frequency relationships for each hanger under the basic traffic condition.

The inclination of No. 139 is steeper than that of other four hangers in high stress region. The sum of the absolute values of the maximum values and the minimum values of the influence line of the hanger No. 139 and that of other four hangers are 0.4 and 0.2 to 0.23, respectively.

Consequently, it is assumed that ordinate of the influence line will effect most on the stress range produced by traffic live loads. Based on the above result, hangers No. 139 and 118 are selected for the fatigue evaluation. The longest hanger No. 118 is selected for the evaluation because the fatigue strength of cables decreases as its length increases.

The stress ranges and frequencies for the fatigue evaluation are set under the following conditions, based on the result obtained by the simulation analysis.

i) The life time of the bridge is supposed to be 100 years. The total stress ranges and frequencies for 100 years are obtained by shifting the line of the stress range-frequency relationships calculated by the simulation analysis in one month to the position corresponding with 100 years.

The upper limit value of the stress ranges is determined by adding the secondary stress caused by bending of the hanger and the maximum stress range of the hanger produced by the design live load.

ii) The stress range, in order to evaluate the fatigue of hangers, is obtained summing up the secondary stress caused by bending of the hanger and the stress range of the axial force. The maximum bending stress generated by the design live load in the longitudinal direction is 6 kgf/mm^2.

iii) The equations of the fatigue strength of HiAm cables proposed in the previous section are expressed by the standard stress range to take the influence of the minimum stress into account.

It is necessary to modify the stress ranges and frequencies to obtain the cumulative damage by this formula. The conversion ratio to the standard stress range is calculated by Eq. (3), by setting the dead load stress as the minimum stress, when the maximum stress range occurs.

The convertion ratio is 1.03 for the hanger No. 139 and 1.13 for No. 118.

(3) Fatigue damage calculation of inclined hangers

For the fatigue evaluation of inclined hangers of Hokko Bridge, both the S-N curves proposed by Birkenmaier[10] and the design S-N curves with the 95% of reliability proposed in this study are used.

Fig. 16 shows the stress range-frequency relationships of No. 139 and No. 118, and the design S-N curves.

Table 4 shows the calculation result of the cumulative damages of No. 139 and No. 118 based on Miner's rule. Cumulative damages are less than 1, and it guarantees the safety of these hangers even if the average weight of heavy trucks is multipled by 1.25 or 1.5. The proposed S-N curve showes that the stress range which gives the cumulative damage equal to that in Table 4, on the hanger No. 139 in basic condition, is 21.5 kgf/mm^2 when the repetition is 2×10^6. The stress range 21.5 kgf/mm^2 obtained here is 55% of the design maximum stress range 38.9 kgf/mm^2 (including the secondary stress). It is equivalent to the use of 55% of design live load.

The method mentioned in this study is able to figure out the fatigue design load of cables as the ratio to the design live load for individual structural and traffic properties, while in DIN 1073 the fatigue design load is always 50% of maximum design live load.

6. CONCLUSIONS

The results of evaluation in this paper are as follows :

(1) In Nanko Ohashi Bridge, where a lot of heavy trucks pass over, the stress ranges and frequencies measured at the main girder can be divided into two regions, high stress range and low stress range and it is also expressed by two straight lines in which the frequency is expressed in logarithm.

(2) The inclination of the stress range-frequency relationships becomes stable when the measured time exceeds one week, and the stress ranges and frequencies for a long term can be estimated by transferring it in parallel to the measured time.

(3) The stress ranges and frequencies can be rather accurately reproduced by the simulation analysis considering both cases of the smooth traffic and the traffic jam.

(4) The fatigue strength of cables may be effectively evaluated by using the results of the fatigue test and the statistical theory of extremes. The S-N curve can be obtained considering the influence of the cable length and the wire diameter, etc.

(5) Applying Miner's rule for cumulative damage, the inclined hangers of Hokko Bridge are safe enough even if the influence of the secondary stress is taken into account.

(6) According to DIN 1073, the specified ratio of the fatigue design load to the design live load is 50 %. However, on the basis of the stress ranges and frequencies analysed by the simulation and the S-N curves proposed in this paper, the ratio at 2×10^6 cycles is 55%.

REFERENCES

1) Flint & Neill Partnership Consulting Engineers : M 4 Severn Crossing, Structural Feasibility Study, Final Report, Vol. 1, 5, 1983.
2) Fujino, Y., Itoh, M. and Endoh, G. : Design Traffic Live Load for Highway Bridge Based on Computer-Simulation, Proc. of JSCE, No. 286, June, 1979.
3) Investigation of the Live Load of Hanshin Expressway and its Analysis for Load Evaluation, Hanshin Kohsoku Doro Kodan, Reports by Design Load (HDL) Committee, Mar., 1984.
4) Miki, C., Goto, Y., Yoshida, H. and Mori, T. : Computer Simulation Studies on the Fatigue Load and Fatigue Design of Highway Bridges, Proc. of JSCE, Structural Eng./Earthquake Eng. Vol. 2, No. 1, 4, 1985.
5) Makino, F., Komatsu, S., Okada, Y. and Kubo, M. : Reliability Assessment for Fatigue Resistance of Cable under Stochastic Live Loads, Proc. of JSCE, No. 362, Oct., 1985.
6) Shinke, T., Hironaka, K. and Oishi, Y. : Fatigue Strength of PWS, R & D, Kobe Steel-Making technical Report, Vol. 28, No. 2, Feb., 1978.
7) Tanaka, Y. and Haraguchi, T. : Fatigue Characteristics of HiAm Anchor Cable, Proc. of the 34 th Annual Conference of JSCE, I-56, Oct., 1979.
8) Sub-committee for the study of Steel Superstructure of the Honshu-Shikoku Bridge, Section Meeting for Cable, Mar., 1978.
9) Andrä, W. and Saul, R. : Versuche mit Bündeln aus parallelen Drahten und Litzen fur die Nordbrücke Mannheim-Ludwigshafen und das Zeltdach in München, Die Bautechnik, 9, 10, 11, 1974.
10) Birkenmaier, M. : Fatigue Resistant Tendons for Cable-Stayed Construction, IABSE Periodica 2, 1980.
11) Andrä W. and Saul, R. : Die Festigkeit insbesondere Dauerfestigkeit Langer Paralleldrahtbündel, Die Bautechnik, 4, 1979.
12) Castillo, E., Fernandez Canteli, A., Esslinger, V. and Thurlimann, B. : Statistical Model for Fatigue Analysis of Wires, Strands and Cables, IABSE Proceedings P-82/85, 2, 1985.
13) Matsukawa, A., Kamei, M., Fukui, Y. and Sasaki, Y. : Fatigue Analysis of Parallel Wire Strand Cables under Consideration of Influencing Factors, Journal of Structural Engineering, Vol. 31 A, March, 1985.

14) Matsukawa, A., Kamei, M., Mizoguchi, T. and Sasaki, Y.: Fatigue Strength Analysis of Parallel Wire Strand Cables Based on the Statistical Theory of Extremes, Journal of Structural Engineering, Vol. 32 A, April, 1986.
15) DIN 1073: Stählerne Straβenbrücken Berechnunggrundlagen, 7, 1974.
16) Matsukawa, A., Kamei, M., Yamamoto, M. and Isoda, A.: Traffic Simulation Analysis for Evaluation of the Fatigue Damage of Inclined Hangers on Hokko Bridge, Proc. of the 40 th Annual Conference of JSCE, I-159, Sep., 1985.
17) Ibukiyama, S.: Road Traffic Engineering, Kanehara Publishing Co., Ltd., 1964.
18) Matsukawa, A., Kamei, M., Yamauchi, T. and Yamamoto, M.: Investigation of the Distribution of Stress Ranges and Frequencies Caused by Actual Live Loads in Osaka Nanko Port Area, Proc. of the 40 th Annual Conference of JSCE, I-158, Sept., 1985.
19) Gumbel, E. J. (Translated & Supervised by Kawada, et al.): Statistics of Extremes, Seisan Gijutsu Center Sinsha, 1978.

(Received December 16 1985)