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Fatigue Reliability Level of Railway Bridges in China

Fiabilité des ponts de chemins de fer en Chine du point de vue de la fatigue

Zuverlässigkeit von chinesischen Eisenbahnbrücken in Bezug auf Ermüdung

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SUMMARY

In this paper, the fatigue reliability level of railway bridges designed by the Chinese Railway Bridge Design Code is calibrated by level II reliability analysis on the basis of available load spectras, laboratory data and field observations of stress histories.

RÉSUMÉ

Le comportement à la fatigue et la sécurité des ponts de chemins de fer, calculés selon les normes chinoises pour les ponts-rails, sont étudiés à l'aide de l'analyse de la fiabilité, et sur la base des cas de charges connus, des données de laboratoire et d'observations sur des ouvrages existants.

ZUSAMMENFASSUNG

Das Zuverlässigkeits-Niveau in Bezug auf Materialermüdung von Eisenbahnbrücken, welche aufgrund der chinesischen Normen bemessen wurden, wird mit Hilfe der Zuverlässigkeits-Theorie überprüft. Dabei werden vorhandene Lastspektren, Labor-Ergebnisse und an Bauwerken beobachtete Spannungs-Geschichten berücksichtigt.

1. INTRODUCTION

In order to contribuite to development of probability based limit state design of railway bridge structures in China, the fatigue reliability level of railway bridges designed by the Chinese Railway Bridge Design Code is calibrated by first-order, second-moment procedure on the bases of available load spectras, laboratory data and fied observations of stress histories.
Accoding to the CRBD Code the bridge structures are designed for fatigue assuming that all cyclic loads cause stress equal to that induced by the maximum live load namely Chinese Railway Live Load shown in Fig.1. If the maximum stress falls above the fatigue limit for constant amplitude cycling the allowable number of cycles, $2\cdot10^6$, for the maximum stress is obtained from an allowable wöhler curve. If it falls below the fatigue limit, the structure is said to have an infinete life.
Field mesurment of actual live load defeine variable amplitude stress range histograms. With the equation of the Wöhler curve, $N=C\Delta\sigma^{-K}$, and the formulas for the comulative damage of the Palmgren-Miner hypothesis, an equivalent stress range of constant amplitude $\Delta\sigma_e$ that produces the same degree of fatigue damage as the variable amplitude stress ranges it replaces.
The fatigue reliability level of bridge structures designed by the present chinese code is calibrated by reliability indices.

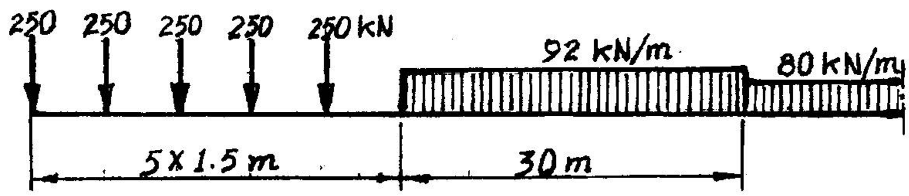

Fig.1. CR design live load

2. DISTRIBUTION OF STRESS RANGES UNDER TRAFFIC LOADS

In order to examine the real traffic conditions on various lines, the stresses under traffic loads were measured on 13 railway bridges. The survey concerning the bridges studied is given in Table 1.
At selected sports of the bridge structure, the stress was recoded by means of strain gauges on magnetic tape for tests of long duration. The stress-time history was further processed in the laboratory.
The recodings show quite regular stress fluctuations under the passage of the passenger train, they are much more random under the freight train. The locomotives of both train types cause predominant stress peaks. The frequency of occurrence of stress ranges is defined by a histogram in which the height of the bar represents the percentage of stress ranges within an interval represented by the width of the bar. The frequency-of-occurrence data can be presented in a more general way by dividing the height of each bar by the width of the bar to obtain a probability density curve.
Let $\Delta\sigma$ be a random variable denotiong stress range. The study of

field data shqwed that the distribution of the applied stress range $\Delta\sigma$ may be conveniently modeled with the beta-distribution. This particular form of the distribution with specified upper and lower limits for the stress range is appropriate. The beta-density curve can be skewed by changing the parameter values. Three types of stress range pattens are suggested, corresponding to "light", "medium" and "heavy" loading conditions, respectively. The statistics of these three stress range distribution used in the calibration are given in Table 2.

Rail-way	Material	span (m)	Bridge compo-nested	Number of days	Number of trains
ZG	Reinforced concrete	16	main girders	2.5	140
SF	Pretensioned concrete	16	main girders	3	120
BB	Pretensioned concrete	31.7	main girders	2	95
HC	Reinforced concrete	4.5	main girders	2	95
NL	Reinforced concrete	16	main girders	4	105
L I	Post-tension-ed concrete	23.8	main girders	2.5	101
L II	Reinforced concrete	8	main girders	2	110
LL	Post-tension-ed concrete	31.7	main girders	3	90
JG	Post-tension-ed concrete	31.7	main girders	2	92
JP	Post-tension-ed	31.7	main girders	2	108
FS	Post-tension-ed concrete	31.7	main girders	2	128
PY	Steel trusses	48	truss chords cross beams stringers	7	490
XG	Steel girders	24	main girders cross beams stringers	7	340

Table 1 Survey of tested bridges

The question arises as to how to treat theoretically such distribution of stress ranges in order to assess the cumulative fatigue damage under the random stress range. The method of equivalent damage has been shown one of the most effective.
Based on the linear damage accumulation hypothesis of Palmgren-Miner and the fatigue strength lines accoding to $N = C\Delta\sigma^{-k}$, the equiva-

lent stress range $\Delta\sigma_e$ can be obtained

$$\Delta\sigma_e = [E(\Delta\sigma^k)]^{1/k} \tag{1}$$

where $E(\Delta\sigma^k) = \int_0^\infty \Delta\sigma^k f(\Delta\sigma) d\Delta\sigma$

and $f(\Delta\sigma)$= the probability density for the random variable $\Delta\sigma$. For a beta-distribution with a maximum value of $\Delta\sigma_{max}$, a minimum of o, and shape parameters q, r, the m-th statistical moment of $\Delta\sigma$ becomes

$$E(\Delta\sigma^k) = \Delta\sigma_{max}^k \cdot \frac{\Gamma(q+r)\Gamma(k+q)}{\Gamma(q)\Gamma(k+q+r)} \tag{2}$$

where $\Gamma(\cdot)$ represents the gamma function. Thus the equivalent stress range for a beta-distributed stress range becomes

$$\Delta\sigma_e = \Delta\sigma_{max}\left(\frac{\Gamma(q+r)\Gamma(k+q)}{\Gamma(q)\Gamma(k+q+r)}\right)^{1/k} \tag{3}$$

Noting that $\Delta\sigma_{max} = (\Delta\sigma_{max}/\Delta\sigma_{max}^c)\cdot\Delta\sigma_{max}^c = \alpha\,\Delta\sigma_{max}^c$, substituting $\Delta\sigma_{max}^c = (\Delta\sigma_{max}^c/\Delta\sigma_{cR})\cdot\Delta\sigma_{cR} = \gamma\,\Delta\sigma_{cR}$ and introducing the equivalent stress factor $\varphi = \Delta\sigma_e / \Delta\sigma_{max}$, we obtain

$$\Delta\sigma_e = \Delta\sigma_{cR}\cdot\alpha\cdot\gamma\cdot\varphi \tag{4}$$

where $\Delta\sigma_{cR}$ = the computed stress range due to design loading;
$\Delta\sigma_{max}^c$ = the computed stress range due to actual traffic loading;
$\Delta\sigma_{max}$ = the measured stress range due to actual traffic loading
α = the ratio of actual traffic to theoretical stress, accounting for stress reduction due to participation of elements disregarded in the design, impact values different from design values, stress dissipation, etc;
γ = the ratio of actual stress range to design stress range;
φ = $(\Gamma(q+r)\Gamma(k+q)/\Gamma(q)\Gamma(k+q+r))^{1/k}$ = equivalent stress factor.

It is to be noted that α, γ and φ can be treated as random variables. From the above formula, mean and coefficient of variation of equivalent stress range $\Delta\sigma_e$ are obtained as follows

$$\overline{\Delta\sigma_e} = \Delta\sigma_{cR}\cdot\bar{\alpha}\cdot\bar{\gamma}\cdot\bar{\varphi} \tag{5}$$

$$V_e = (V_\alpha^2 + V_\gamma^2 + V_\varphi^2)^{1/2} \tag{6}$$

Mean values and coefficients of variation of these random factors were determined from the results of tests, measurements and engineering judgements. These are summarized in Table 3.

loading	q and r of beta-distribution q r	upper limit of stress range $\Delta\sigma_{max}$	mean $\overline{\Delta\sigma}$	standard deviation $S_{\Delta\sigma}$	coefficient of variation $V_{\Delta\sigma}$
Light	1 6	0.7 $\Delta\sigma_{cR}$	0.175 $\Delta\sigma_{cR}$	0.101 $\Delta\sigma_{cR}$	0.577
Medium	4 4	0.7 $\Delta\sigma_{cR}$	0.350 $\Delta\sigma_{cR}$	0.170 $\Delta\sigma_{cR}$	0.334
Heavy	4 1	0.7 $\Delta\sigma_{cR}$	0.560 $\Delta\sigma_{cR}$	0.114 $\Delta\sigma_{cR}$	0.204

Remak $\Delta\sigma_{cR}$=the stress range due to Chinese Railway Live Load.

Table 2 Statistics of stessrange distributions

Loading	$\Delta\sigma_e$	V_α	V_γ	V_φ	V_e
Light	$0.225\ \Delta\sigma_{cR}$	0.15	0.12	0.40	0.444
Medium	$0.387\ \Delta\sigma_{cR}$	0.15	0.11	0.40	0.441
Heavy	$0.581\ \Delta\sigma_{cR}$	0.15	0.10	0.40	0.439

Table 3 Statistics of $\Delta\sigma_e$

3. RANDOM FATIGUE RESISTANCE

Under the condition of a constant stress range, the fatigue life of a structural component or detail has been observed to exhibit considerable variability and, therefore, should be described with a random variable.

Since the data seldom are sufficient to define the probability distributions, we nomally must rely on assumed distributions which arise from relevant physical arguments. The weibull and lognormal distribution have been widely used in fatigue studies.

The results of a series of tests for the same value of $\Delta\sigma$ in the number of load cycles until fracture are assumed to follow a lognomal distribution. Use of the lognormal distribution has been based primarily on arguments of mathematical expelience.

If fatigue strength $\Delta\sigma$ is assumed to be lognormal, the least square line is the median of $\Delta\sigma$ for a given N, denoted as $\widetilde{\Delta\sigma}$. The mean value of $\Delta\sigma$ is estimated as $\ln\overline{\Delta\sigma}$.

The coefficient of variation of $\Delta\sigma$ should incoporate all sources of uncertainty inherent to the fatigue behavior of the structural detail in service. In addition to inherent scatter in the laboratory test data, these would include errors in the stress analysis, the effects of fabrication and workmanship, sampling and measurment.

The value of uncertainty in the fatigue resistance $V_{\Delta\sigma}$ may be evaluated in terms of the individual sources of uncertainty and combined systematically through statistical methods. An estimate for $V_{\Delta\sigma}$ may be obtained by first-order statistical analysis

$$V_{\Delta\sigma} = \frac{1}{K^2}\left(V_c^2 + V_N^2 + V_g^2\right)$$

where V_c = the uncertainty in the intercept of the Wöler curve; this should include the uncertainty in the effect of fabrication and quality of workmanship;

V_N = scatter in the fatigue life data about the Wöler curve;

V_g = the inaccuracy of the fatigue model

The objective in calibration is to check the reliability level in fatigue design covered by the present code. In the calibration the fatigue strength of structural members and details are taken as fatigue resistances. In the fatigue limit state, resistance is expressed in terms of stress ranges. In the CRBD Code calibration several characteristic detailes and members are selected. They are often used in bridge structures constructed in China. In the FOSM methods of reliability checking a random variable is usually characterized by two values, such as the mean and the coefficient of variation. The resistances are characterized by their nominal values, the ratios of mean to nominal, $\overline{\Delta\sigma}/\Delta\sigma_R$, and the coefficient of variation $V_{\Delta\sigma}$.

The values of $\overline{\Delta\sigma}/\Delta\sigma_R$ and $V_{\Delta\sigma}$ used in calibration are calculated on the basis of information provided by various specialized research groups in China, and adopted from engineering judgment. The adopted statistics of fatigue resistance are listed in Table 4.

Detials and mumbers	$\Delta\sigma/\Delta\sigma_n$	K	V_N	$(V_c^2+V_g^2)^{\frac{1}{2}}$	$V_{\Delta\sigma}$
Continuous manual longitudinal fillet welds parallel to the diraction of stress.	1.02	-2.78	0.359	0.4	0.193
Beam flange with cover plate-not welded across plate ends.	1.00	-2.97	0.271	0.4	0.163
Transverse stiffener fillet welded to beam web.	1.01	-2.75	0.509	0.4	0.235
Plate transverse butt welded to beam flange.	1.02	-3.17	0.428	0.4	0.185
Post-tensioned concrete beam.	1.00	-3.50	0.201	0.4	0.167

Table 4. Statistics of fatigue strength

4. RELIABILITY INDICES

In order to conduct calibration of fatigue reliability level of bridge structures designed by present code, which is the purpose of this study, the First-Order, Second-Moment procedure is thought to be appropriate. In the FOSM procedure the degree of reliability is characterized by a reliability index which is defined, in general, as

$$\beta = \frac{\bar{Z}}{\sigma_z} \tag{7}$$

in which $Z = g(X_1, \ldots, X_n)$ is the fomulation variable correspon-ding to the limit state $g(X_1, \ldots, X_n)=0$, X_i are the basic random variables in the formulation of the problem; $\bar{Z}$ and σ_Z are the mean and the standard deviation of Z, respectively.

The reliability index for fatigue may be obtained by formulating the limit state

$$Z = \ln(\Delta\sigma/\Delta\sigma_e) = 0 \tag{8}$$

where $\Delta\sigma$ = the fatigue strength and $\Delta\sigma_e$ =the equivalent stress range. In practice the mean and standard deviation of Z can be takan as the first-order approximations

$$\bar{Z} = \ln(\overline{\Delta\sigma}/\overline{\Delta\sigma_e}) \tag{9}$$

$$\sigma_Z = (V_{\Delta\sigma}^2 + V_e^2)^{\frac{1}{2}} \tag{10}$$

where $\overline{\Delta\sigma}$, $V_{\Delta\sigma}$ =mean and coefficient of variation of fatigue strength, and $\overline{\Delta\sigma_e}$, V_e =mean and coefficient of variation of equi-valent stress range, respectively. Thus the reliability index is given by

$$\beta = \frac{\ln\left(\frac{\overline{\Delta\sigma}}{\Delta\sigma_e}\right)}{\left(V_{\Delta\sigma}^2 + V_e^2\right)^{1/2}} \qquad (11)$$

The typical details of welded plate girders and prestressed concrete girders of railway bridges designed according to the CRBD Code were calibrated using Eq. (11). The calculated values of the fatigue reliability indices β are given in Table 5.

Category of bridge Described in table 4	Total uncertainty $(V_{\Delta\sigma}^2 + V_e^2)^{1/2}$	Traffic load factor $\overline{\Delta\sigma_e}/\Delta\sigma_{eR}$ Light	Medium	Heavy
		0.225	0.387	0.581
1	0.484	3.58	2.48	1.64
2	0.473	3.63	2.49	1.62
3	0.502	3.49	2.36	1.55
4	0.481	3.59	2.47	1.62
5	0.474	3.66	2.53	1.67

Table 5 Calculated values of the reliability indices β

5. CONCLUSIONS

The primary objective of this paper is to check for the fatigue reliability level of actual traffic on railway bridges designed by the present CRBD Code. As a result, it is shown that the ranges of reliability indices are found to be from 1.5 to 3.6 for fatigue design situation. The level of is affected very much by coefficients of variation of loads and resistance, especially if they are large.

Other design situations such as the ultimate limit state and the second serviceability limit state were not considered in this calibration collection of statistical data of loads and resistance of bridge structures and calibration of them with reliability indices are strongly needed in China for future probability based limit state design of railway bridge structures.

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