Bridge load models for fatigue

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Modèles de charge de fatigue pour des ponts en acier

Lastmodelle zur Untersuchung der Ermüdung von Stahlbrücken

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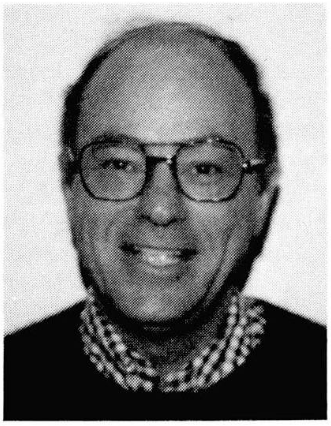

Fred Moses received a civil engineering degree from CUNY in 1960 and a PhD from Cornell in 1963. Since joining Case Institute, Professor Moses has been active in structural optimization and reliability with special applications to bridge and offshore structures including loading, safety analysis and system reliability studies.

SUMMARY

This study was performed to support new specifications for fatigue life design of steel bridges and also, the assessment of remaining fatigue life of existing steel bridges. A major part of the research included modelling truck loading and bridge stress response. Data was collected on truck weight distributions, dynamic response, lateral distribution analysis and stress spectra. A load model is proposed in order to predict bridge response. Data is used to calibrate the load model by describing each variable in terms of its coefficient of variation. An application of the load model to fatigue specifications is presented.

RÉSUMÉ

Cette étude a été effectuée dans le but de soutenir de nouvelles directives relatives à la conception à la fatigue de nouveaux ponts en acier, ainsi qu'à l'évaluation de la durée de vie restante des ponts existants. Une grande partie de la recherche a été consacrée au développement des modèles de charge de camion et à l'analyse des contraintes dans l'ouvrage. Des statistiques sur la distribution du poids des camions, ainsi que sur la réaction dynamique, la répartition transversale et les spectres de contrainte ont été rassemblées. Un modèle de charge est proposé dans le but de pouvoir prédire la réponse de l'ouvrage. Les données statistiques ont été utilisées pour établir ce modèle de charge en considérant le coefficient de variation de chacune des variables. Un exemple d'utilisation du modèle de charge pour l'étude de la résistance à la fatigue est présenté.

ZUSAMMENFASSUNG

Diese Studie wurde mit dem Ziel durchgeführt, neue Vorschriften über die Ermüdungsbemessung von Stahlbrücken und die Schätzung der Restlebensdauer bestehender Stahlbrücken aufzustellen. Der grösste Teil der Forschungsarbeit betrifft die Entwicklung von Lastmodellen zur Berücksichtigung des Schwerverkehrs und das Studium der in Brückenkonstruktionen auftretenden Spannungen. Daten wurden gesammelt zur statistischen Verteilung der Fahrzeuggewichte, zum dynamischen Verhalten von Brücken, zur Querverteilung der Lasten sowie zu den in Brücken vorkommenden Spannungsspektren. Ein Lastmodell wird vorgeschlagen, um die Reaktion einer Brücke vorhersagen zu können. Zur Kalibrierung dieses Modells wurden die Variationskoeffizienten aller vorgenannten Variablen verwendet. Ein Beispiel zeigt, wie das Lastmodell zur Untersuchung von Ermüdungsproblemen angewendet werden kann.

1. INTRODUCTION

The maintenance and safety of existing bridges is an important concern of all highway agencies. To assure adequate safety and to assist in assessing maintenance needs, highway bridges are periodically inspected, usually at 2-year intervals in the United States. In conjunction with such inspections, a safety rating is established by procedures given in the AASHTO *Manual for Maintenance Inspection of Bridges* [1]. The *Manual* presents detailed procedures for rating the strength capacity of steel bridges but does not give detailed procedures for assessing the safety with respect to fatigue. Instead, it suggests that the AASHTO *Standard Specifications for Highway Bridges* [2] be used as a guide in assessing fatigue strength.

The fatigue provisions in the *Specifications* were originally adopted before adequate information was available on the fatigue conditions in actual bridges. Therefore, these provisions do not reflect the fatigue conditions that actually occur. Instead, they combine an artificially high stress range with an artificially low number of stress cycles to produce a reasonable design. Furthermore, the fatigue provisions in the *Specifications* are presented in terms of allowable stresses and do not indicate how to calculate the remaining life of an existing bridge, which is needed in decisions regarding inspection, repair, rehabilitation, and replacement.

The objective of the present study is to develop practical fatigue evaluation procedures that:

1. Realistically reflect the actual fatigue conditions in highway bridges
2. Give an accurate estimate of the remaining fatigue life of a bridge and permit this estimate to be updated in the future to reflect changes in traffic conditions.
3. Provide consistent and reasonable levels of reliability.
4. Permit different levels of effort to reduce uncertainties and improve predictions of remaining life.
5. Apply consistently to both the evaluation of existing bridges and the design of new bridges.
6. Can be conveniently modified to reflect future research.
7. Are suitable for both the AASHTO Maintenance Manual [1] and Design Specifications [2].

2. FATIGUE LOAD MODEL

Fatigue life of steel bridges is primarily affected by the repetitive load cycles caused by passage of heavy vehicles. The stress ranges that occur at critical locations in actual bridges under normal traffic have been extensively measured. Results are usually reported in terms of effective stress range which is defined as the equivalent constant-amplitude stress range that provide the same fatigue damage as the variable-amplitude spectrum. The average effective stress range for some 215 histograms surveyed is 1.8 ksi [3]. The single highest effective stress range is 4.9 ksi and the next highest is 4.4 ksi. These data suggest that the peak stress range for histograms for steel girder or beam bridges is almost always below 10 ksi and that the effective stress range is almost always below 4.5 ksi (1 ksi = 6.89 Mpa).

Design stress ranges calculated by present AASHTO procedures are usually well above these measured stress ranges. Many factors contribute to the difference. Some of these result from the use in fatigue calculations of static-design procedures that are based on extreme conditions. Fatigue damage actually results from typical, or average, rather than extreme conditions. Specifically, the AASHTO live load design truck, the AASHTO lateral distribution factors, and the AASHTO impact factors are too conservative for fatigue calculations.

Many other factors that contribute to the difference between design and measured stresses are difficult to calculate and are conservatively ignored in design calculations. These include (1) unintended composite action; (2) contributions to strength from nonstructural elements, such as parapets; (3) unintended partial end fixity at abutments; (4) catenary tension forces due to "frozen" joints or rigid end supports; (5) longitudinal distribution of moment; (6) direct transfer of load through the slab to the supports; and (7) direct transfer of load through the deck to supports in truss bridges. Although these factors are difficult to calculate, they consistently combine to produce actual stresses well below those calculated by normal procedures.

Average daily traffic volumes (ADT) of 120,000 are not unusual on major six-lane highways in large cities. The corresponding traffic volume in one direction is 60,000. About 10 percent of urban traffic is composed of trucks and about 75 percent of these trucks are in the shoulder lanes. Thus, the average daily truck traffic (ADTT) in the shoulder lane may exceed 4,500 in some cases. This truck volume applied over a 50-year life results in 82 million truck passages. Many bridges put into service in the 1930's are now 50 years old. In most cases, each truck passage causes one stress cycle, but in some cases it may cause more equivalent stress cycles. Thus, well over 100 million stress cycles, and perhaps as high as 300 million cycles, can occur in some bridges. The cyclic life categories used in selecting the AASHTO allowable fatigue stresses are generally well below this number, for example, 2 million cycles [2]. Thus, comparing the true situation to the specification shows lower effective stresses but a much higher number of cycles.

2.1 Concepts of Fatigue Safety

The concept of safety as applied to repetitive loads that cause fatigue damage is quite different from the concepts that are applied in strength design of a bridge with respect to maximum static (nonrepetitive) loads. For fatigue, many loading repetitions are required to produce a failure at some time in the future. Generally, all truck loading stresses, whether above or below the allowable stress range value, cause fatigue damage. The effects of fatigue loading on an existing bridge can best be defined in terms of the remaining safe fatigue life of the bridges. Similarly, the effects of fatigue loading on a new bridge can best be defined in terms of the total life of the bridge, although it may be convenient to use a permissible stress range corresponding to a desired design life to facilitate the reproportioning of members that do not have an adequate life. Safety factors can be applied in calculating the remaining or total life to assure that the actual life will exceed the calculated life with a desired degree of reliability.

2.2 Proposed Design or Evaluation Procedures

Several techniques have been proposed for fatigue design or evaluation procedures for various structural applications. These procedures are intended to realistically reflect the actual fatigue conditions that occur in the structure under consideration. Consequently, they generally involve three steps: (1) calculate the variable-amplitude stress spectrum caused by the actual loading, (2) relate this variable-amplitude stress spectrum to an equivalent or effective constant-amplitude stress by some cumulative damage approach, and (3) compare the resulting applied stress parameter with a fatigue strength (SN) curve to obtain the fatigue life. In many of the design procedures, probabilistic methods are used to define the degree of uncertainty in the calculations and to provide consistent levels of safety. These approaches are consistent with the probabilistic or reliability approaches used in various strength design codes. Factors of safety may be applied to the applied stress, the strength parameter and/or the design life.

The uncertainty in the calculated variable-amplitude stress spectrum depends on how accurately the loading can be defined and on how accurately stresses can be calculated from the loading. For highway bridges, truck traffic is the main fatigue loading and a gross weight histogram for such traffic is the most important parameter defining this loading. Therefore, most of the proposed procedures define a typical histogram or permit the use of an actual histogram for the site. The wheel spacings and distributions of the gross weights to various axles are also important and are defined in most of the proposed procedures. Calculation of the stresses from the loading involves factors to account for lateral distribution, impact, and truck superpositions. These parameters are usually covered in some way in the proposed procedures.

Miner's linear cumulative damage law is used in almost all of the proposed procedures to relate variable-amplitude fatigue behavior to constant-amplitude behavior derived from lab test data. Although Miner's Law is often criticized by researchers, especially those dealing with special types of loadings, an extensive study of simulated bridge members showed that it is unbiased and that the scatter in predicting the life is not large. In this concept, a variable-amplitude spectrum is represented by an equivalent constant-amplitude stress cycle. This concept is carried one step further herein and uses an effective fatigue truck to represent typical truck traffic.

3. LOAD MODEL

A consistent load model is needed for fatigue design and evaluation procedures. Each variable must be identified and test data used to estimate the bias (mean to nominal value) and coefficient of variation (COV). Because the fatigue process is an averaging of damage accumulation, it is important that scatter emphasize site to site variation. For example, dynamic impact varies greatly with different vehicle types and gross weights. Hence, measured impacts at a site will have a large COV. But for that particular site the damage accumulation will not reflect this large scatter since the average damage per vehicle is of concern in computing the life. Thus, the mean impact is of greatest interest. The variation of this mean impact, however, from site to site is important since it greatly affects the estimate of accumulated damage per unit time interval.

3.1 Load Calculation

The true stress for any truck crossing depends on several variables and may be written as:

$$S_i = \frac{M_i}{S_x} = \frac{W_i gimh}{S_x} \qquad (1)$$

where S_i is the nominal stress range on the attachment detail, M_i = maximum bending moment range of the ith truck crossing event on the girder, W_i = ith truck crossing gross vehicle weight, m = influence factor relating truck weight to maximum bending moment, i = impact amplification, g = lateral girder distribution (expressed as percent of gross span moment carried by a single member), h = account for closely spaced or multilane presence of vehicles which amplify the moment, and S_x = actual section modulus.

Equation 1 gives the stress range for the ith truck crossing event. It relates to the design stress range through the selection of the section modulus, which is expressed using similar terms:

$$S_{XD} = \frac{\gamma\, W_D g_D i_D m_D h_D}{S_{rD}} \tag{2}$$

where γ is the reliability factor to be specified after calibration of the risk (this factor ensures an acceptable risk for the computed fatigue life of the member). W_D, g_D, i_D, m_D, h_D, are the specified nominal or design values of W, g, i, m, and h, respectively. S_{rD} is the design or allowable stress range. (This is obtained from the fatigue strength curves (SN curves) corresponding to expected lifetime total number of cycles the member is subjected). S_{XD} is the computed value of section. In general, the actual section modulus S_X is a random variable. S_{XD} and S_X are related by a random variable Z_X by:

$$S_X = Z_X S_{XD} \tag{3}$$

Z_X reflects the scatter in the true section modulus compared to nominal section modulus.

3.2 Damage Accumulation

According to Miner's rule the accumulated damage is:

$$D = \Sigma \frac{1}{N(S_i)} \tag{4}$$

where the sum is over each of the stress cycles S_i. $N(S_i)$ is the number of cycles to failure at a constant amplitude stress range of S_i. The fatigue curve may be described as the straight line log S vs. N curve written as:

$$NS^b = c \tag{5}$$

where the exponent b equals 3 for most welded attachments [3].

Substitution in Eq. 4 gives:

$$D = \frac{1}{c} \Sigma\ S_i^3 \tag{6}$$

Substituting Eqs. 1, 2, and 3 into Eq. 6 gives:

$$D = \frac{1}{c} \Sigma\left(\frac{W_i gmih}{\gamma Z_x W_D g_D i_D m_D h_D} S_{rD}\right)^3 \tag{7}$$

$$= \frac{V}{c} \left(\frac{gmih}{\gamma Z_x W_D g_D i_D m_D h_D} S_{rD} \right)^3 \Sigma \frac{W_i^3}{V}$$

where V = number of trucks per year.

To simplify, let

$$W_{eq} = (\Sigma f(W_i) W_i^3)^{1/3} \tag{8}$$

where W_{eq} = equivalent fatigue truck weight, and $f(W_i)$ = percentage of trucks within weight interval W_i. Let:

$$V = ADTT(365)C \tag{9}$$

where V = volume, reflecting the total number of equivalent stress cycles in a year (a random variable), ADTT = average daily truck traffic in vehicles per day (a random variable), C = equivalent number of stress range cycles per truck crossing (a random variable).

Substituting Eqs. 8 and 9 into Eq. 7 gives:

$$D = \frac{1}{c} \quad ADTT(365)C \quad \left(\frac{W_{eq} gmih}{\gamma Z_x W_D g_D i_D m_D h_D} S_{rD} \right)^3 \tag{10}$$

Simplifying further gives:

$$c = N_T S_r^3 = N_T \quad \left(\frac{S_r}{S_{rD}} \right)^3 \quad S_{rD}^3 \tag{11}$$

where S_r is the true stress range (a random variable) from the SN curve corresponding to N_T number of stress cycles. N_T is a reference number of cycles and is deterministic. For convenience, it is chosen to be numerically equal to the total number of expected stress cycles in the lifetime of the member.

From the definition of N_T,

$$N_T = \overline{ADTT}(365)\overline{C}\, Y_s \tag{12}$$

where $\overline{ADTT}$ and $\overline{C}$ denote the mean values of random variables ADTT and C. Substituting Eqs. 11 and 12 into Eq. 10 gives:

$$D = \frac{ADTT(365)C}{N_T} \quad \left(\frac{W_{eq} gmih}{\gamma Z_x W_D g_D i_D m_D h_D} \; \frac{1}{Z_x} \; \frac{S_{rD}}{S_r} \right)^3 \tag{13}$$

Let:

$$G = g/g_D \tag{14}$$

$$I = i/i_D \tag{15}$$

$$M = m/m_D \tag{16}$$

$$W = W_{eq}/W_D \tag{17}$$

$$H = h/h_D \tag{18}$$
$$S = S_r/S_{rD} \tag{19}$$

Equations 14-19 define as random variables the ratio of the true value (random) to the nominal design value.

Substitution now gives:

$$D = \frac{ADTT(365)C}{N_T} \left(\frac{W\,G\,I\,M\,H}{\gamma Z_x S}\right)^3 \tag{20}$$

Letting the damage accumulation per year be denoted as D, we can express the random variable, Y_F which is the life at which failure actually occurs.

$$Y_F = \frac{X}{D} \tag{21}$$

where X is a random variable accounting for model uncertainty (mainly Miner's law assumption).

Substituting Eqs. 20 into 21 we have:

$$Y_F = \frac{XN_T}{365(ADTT)C} \left(\frac{\gamma Z_x S}{W\,G\,I\,M\,H}\right)^3 \tag{22}$$

The random variables included in the fatigue life expression contain material terms, Z_X, X and S, truck variables, W, ADTT, H, M, and C and analysis uncertainties I and G. The function given by Eq. 22 is the input to a reliability program. This input must also include statistical parameters and distribution functions of each of the 10 variables discussed in the next section.

4. DATA BASE

In the previous section, the random fatigue life is expressed in terms of random material, loading and analysis variables. Data was accumulated on each of these variables to estimate a nominal value and bias ratio, COV and distribution function. These serve as input to a reliability model to provide consistent design and evaluation fatigue procedures for steel bridges. The data is reviewed herein from a recent report [3]. The data was used to calibrate the reliability procedures by: a) computing safety indices for a range of existing bridges, b) extracting a target safety index, c) selecting a nominal fatigue design and evaluation format and corresponding safety factors (γ) which provide uniform and consistent safety indices over all applications. The same data base is used both in the reliability calculation of existing performance and the selection of the new format values. Hence, small errors in the data base have little or no influence on the final calibration. This would not be the same situation if economic criteria were used to establish the safety indices. Each variable will be separately considered with an assumed lognormal distribution.

4.1 Random Variable M

Although some codes have adopted several of fatigue vehicles, it was decided for simplicity to use only a single fatigue design vehicle. The random variable M, called the moment ratio, reflects the effect of axle spacing and axle weight distribution and is the ratio of the average influence factor due to actual truck spectrum on the bridge to the influence factor of the fatigue design vehicle. Moment ratio varies with span. Truck traffic data from 12 sites were used to calculate the average moment ratios. Data from 5 sites were used to calculate moment ratios for continuous spans. It was found for both simple and continuous spans that the mean or bias of the proposed fatigue design vehicle was reasonably uniform and the COV was about 3%.

4.2 Random Variable W

The random variable W reflects the uncertainty in the estimation of the gross weight of the equivalent fatigue truck. The 54 kip gross weight is obtained from weigh-in-motion (WIM) studies including 30 nationwide sites with over 27,000 truck samples. The value of gross weight is assumed now to be unbiased, although there may be some future growth. A COV of 10 percent is used for the gross weight of fatigue truck in the reliability analysis which implies that there is a 95 percent chance that the effective gross weight of the truck spectrum at a given site will be between 43 kip and 65 kip. This assumption agrees with the results of WIM studies, where effective gross weights in the range of 45 kip to 67 kip were found.

4.3 Random Variable H

The random variable H reflects the effect of multiple presence of trucks on the bridge. Simulating this factor using WIM traffic data for free flow gave a mean value of 1.03 and COV of 0.6 percent.

4.4 Impact Ratio I

The random variable I reflects the uncertainty in estimating the average impact factor for a given site. A mean value of 1.0 and a COV of 11 percent is based on data [3]. Variations from truck to truck within the same site are not important because fatigue is an averaging process and such variations almost cancel out.

4.5 Lateral Distribution Ratio G

The random variable G reflects the uncertainty in the estimation of girder lateral distribution factor. The proposed procedures use the best estimate of the distribution factors. The mean of G is taken as 1.0 with a COV of 13 percent from site-to-site data collected [3].

4.6 Random Variable Z_x

The random variable Z_x reflects the uncertainty in the effective section modulus. In general, the proposed procedures recommend the best estimate of the actual section modulus and, hence, the mean value of Z is taken as 1.0. However, in some specific cases, it is recognized that the effective section modulus is significantly above the actual section modulus because of beneficial effects not normally calculated in design. For example, the effective section modulus is about 15 percent above the actual section modulus for composite sections. Such specific cases are taken care of by increasing the computed section modulus. A COV of 10 percent is assumed for the random variable Z_x.

4.7 Random Variable C

The random variable C represents the equivalent number of stress cycles due to a single truck passage on the bridge. The proposed specification uses the best estimates of C determined from rainflow methods. The coefficient of variation for C, is estimated to be 5 percent.

4.8 Random Variable ADTT

The random variable ADTT represents the true value of the lifetime average daily truck traffic in the shoulder lane. The procedures recommend that the ADTT should be estimated from a site conditions and should be unbiased. Volume uncertainty affects the safety factors much less than stress uncertainty because fatigue damage is linearly proportional to volume but is proportional to the cube of stress range. A 10% COV is used for volume.

4.9 Random Variable S

The random variable S reflects the uncertainty in the estimation of fatigue strength curves. The statistical properties of the random variable S depend on the fatigue category and are obtained from the test results reported from Lehigh University. The mean values and COV of stress ranges at 2 million cycles for different detail categories have been obtained and were used [3].

4.10 Random Variable X

The random variable X reflects the uncertainty in the damage model, mainly due to Miner's Rule. The damage predicted by Miner's Rule is assumed herein to be unbiased. To account for possible test scatter with this rule, a coefficient of variation of 15 percent is used. This value had to be estimated because data on the accuracy of Miner's Rule for welded steel structures were insufficient. This implies there is a 95 percent probability that the predicted life of a specimen using Miner's damage rule will be within 70 percent to 130 percent of the actual life.

5. APPLICATION OF FATIGUE-LIFE RELIABILITY MODEL

In developing and utilizing design procedures, it is normal and appropriate to make conservative assumptions at each step. Many of these conservative assumptions are hidden in various specification parameters and equations. The conservative assumptions are intended to account for uncertainties in each step of the design process by using the most conservative value that could reasonably be expected to occur in that step. Of course, it is highly unlikely that the values for all steps will be at their worst in the same bridge. One of the most important benefits of a reliability analysis is that it shows the interrelationship of the various conservative assumptions that are made at each step in a design or evaluation procedure. For example, in a fatigue evaluation, a larger that expected truck loading may be counteracted by a smaller than expected lateral distribution factor so that the actual life will still exceed the predicted life. Another view of this same analysis is that the overall safety (or fatigue life) can be assured with a very high degree of certainty (say, with a risk of failure of only 10^{-4}) even though the value of the variable in each step is known with much less certainty (say, a 10^{-2} probability level) provided the interrelationship of all variables is properly accounted for in the reliability model. This has an important impact on the overall required safety factor as well as the amount and quality of statistical data needed to produce estimates with high confidence.

The fatigue-life (remaining life) model described above has been applied to many bridge examples . Both the mean life and a safe remaining life based on specified reliability levels have been determined. The calibration of the appropriate safety factor to achieve the described reliability level is discussed in another paper [4]. The remaining life calculation can be applied in several ways to existing bridges to achieve adequate safety. If an evaluation of an existing bridge reveals that the calculated remaining fatigue life is less than desired, the engineer has four options. First, he could recalculate the remaining life using more accurate data. For example, he could use more accurate calculations of lateral distributions or make a traffic survey to obtain site-specific data on truck volume and weight distribution. Second, he could restrict the weight and/or volume of trucks to increase the fatigue life. Third, he could modify the bridge to improve its fatigue life by a retrofit to improve its fatigue characteristics. Fourth, he could institute periodic inspections to assure that fatigue cracks could be detected before components actually failed. Estimates of the remaining lives of various details have proven helpful in selecting appropriate inspection intervals and allocating inspection efforts.

6. CONCLUSIONS

The reliability model described herein was utilized in developing two recent Guide Specifications adopted by AASHTO [5,6]. One concerns the design of new bridges and the second describes the evaluation of safe lives of existing steel bridges. The derivation of the safety factors is discussed in another paper at the conference [4]. An accurate prediction of the remaining fatigue life of a bridge has important uses. Such an estimate is needed in bridge management systems that are used to make decisions regarding inspection, maintenance, repair, rehabilitation, and replacement of existing bridges. Estimates of remaining fatigue life would also be very useful in assessing the effects of permitting a certain class of overloaded vehicles to use the highways. An example of evaluating the effect of proposed truck weight legislation on fatigue costs is also discussed in the other paper [4].

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