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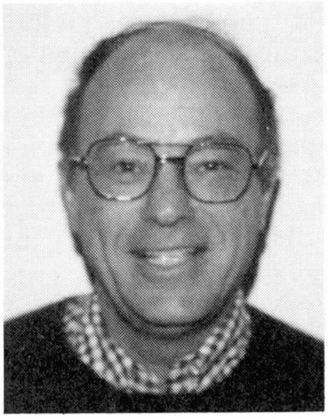
Safe Life Evaluation of Existing Bridges

Evaluation conservatrice de la durée de vie des ponts existants

Bestimmung der sicheren Lebensdauer bestehender Brücken

Fred MOSES
Prof. of Civil Eng.
Case Western Reserve Univ.
Cleveland, OH, USA



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

Estimates of the safe remaining life of existing steel bridges affect decisions on inspection, maintenance, replacement, truck weight, permit posting limits and even changes in new weight regulations. A reliability model has been developed and calibrated to produce uniform and consistent fatigue checking procedures for different spans, geometrics, bridge types, and traffic. These methods have been adopted in recent AASHTO Guide Specifications for fatigue design of new bridges and safe-life evaluation of existing steel bridges. The same model has been extended to compute costs associated with proposed truck-weight regulations. These costs affect new designs and particularly, existing estimates of bridge repair and damage.

RÉSUMÉ

L'évaluation de la sécurité des ponts en acier existants influence les décisions relatives à l'inspection, la maintenance, le remplacement, le poids des camions, les limites de charge à imposer, et peut même engendrer des modifications dans les lois réglant le poids des camions. Un modèle de charge a été développé et calibré afin de conduire à des vérifications à la fatigue uniformes et cohérentes pour différentes portées, géométries, types de ponts et trafics. Ces méthodes ont été adoptées dans les nouvelles recommandations AASHTO pour la conception à la fatigue des nouveaux ponts et l'évaluation de la sécurité des ponts en acier existants. Ce même modèle a été étendu afin d'estimer les coûts liés aux propositions de règlements pour le poids des camions. Ces coûts influencent la conception actuelle et, plus particulièrement, les estimations des réparations et du dommage de ponts existants.

ZUSAMMENFASSUNG

Die Abschätzung der sicheren Restlebensdauer bestehender Stahlbrücken beeinflusst Entscheidungen betreffend Inspektion, Unterhaltung oder Ersatz einer Brücke, betreffend signalisierte Gewichtslimiten oder Änderungen der gesetzlichen Gewichtsbeschränkungen. Ein Modell zur Bestimmung der Ermüdungssicherheit von Stahlbrücken wurde entwickelt. Es wurde so angepasst, dass es in einheitlicher Art und Weise für verschiedene Brückentypen, Spannweiten, Geometrien und Verkehrsmischungen anwendbar bleibt. Die Methode wurde in den kürzlich erschienenen AASHTO-Empfehlungen für die Ermüdungsbemessung neuer Brücken und die Bestimmung der sicheren Lebensdauer bestehender Stahlbrücken übernommen. Dasselbe Modell wurde mit dem Ziel erweitert, die Kosten zu berechnen, welche durch Änderungen der Gewichtsbeschränkungen verursacht werden. Betroffen durch solche Änderungen sind die Kosten neuer Konstruktionen und, in vermehrtem Masse, die Reparaturkosten bestehender Brücken.

1. INTRODUCTION

A companion paper describes a reliability-oriented load prediction model for the random fatigue damage accumulation per year [1]. This damage was expressed in terms of traffic, material and analysis random variables. The data base was expressed in terms of nominal values and bias and respective coefficients of variation. The study described in this paper utilizes the probabilistic damage accumulation and data base to determine safety indices for fatigue failure modes. These safety indices are used to calibrate fatigue procedures for evaluation. These procedures contain nominal checking procedures and factors of safety for both redundant and nonredundant load path structures. The aim is to have uniform and consistent safety indices over the full range of traffic spectra, bridge types and spans. These methods have been incorporated into two recently adopted AASHTO Guide Specifications for design of new steel bridges and safe life evaluation of existing steel bridges [2,3]. Assessment of safe remaining fatigue life is useful for developing inspection intervals, scheduling repair and replacement and making permit decisions. A second application of the fatigue risk model is the extension to a cost allocation study of new truck weight regulations. Projected increases in truck weight and volume changes affect the cost of fatigue damages. Projections were made using the reliability model to the remaining life and costs for over 70,000 steel bridges in the United States over the coming 50-year period [4].

2. FATIGUE RELIABILITY MODEL

This section formulates a reliability approach to predict that the fatigue life of steel beam attachments will be less than the estimated life. Several points to note are: (1) the assumptions of the model, (2) the random variables in the true fatigue life, and (3) the sensitivity of the risk to the statistical parameters of these random variables. The safety margin is expressed in terms of a failure function, g, as in usual structural reliability practice.

$$g = Y_F - Y_S \tag{1}$$

Failure does not occur if $g < 0$. Y_F is life at which failure occurs (a random variable), and Y_S is specified life (deterministic). The expression for Y_F is given in the other paper.

2.1 Safety Index

The safety index (or beta) is the number of standard deviations between the mean of g and the boundary of the safe value of g. If g is normal, the risks corresponding to β are found from a normal probability table, e.g. $\beta = 3$ gives a risk of 0.001. Beta typically falls in the 1 to 4 range in most structural applications. If the variables are not normal, computer programs are available to compute β for general distributions of random variables and failure functions g. The programs output the safety index, β, and the most likely failure values. The latter helps assess in the failure event each random variable. The accuracy in characterizing the statistics of any single variable will decrease as the total number of variables increase as, say, in the fatigue model which has ten random variables. Thus, if we seek a risk of 10^{-4}, the design point value for each variable may fall in only the 10^{-2} range. Thus, realistic and accurate assessments of reliability can be made without requiring an unrealistic amount of data.

2.2 Calibration

Code writers have a responsibility in selecting a consistent target safety index, or β, for a code check. One approach is to base the decision on economics, i.e., an optimum failure rate occurs when the cost trade-off of increasing the safety factor balances the risk reduction of future failures. A problem here is expressing the cost of failure. Another approach is to compile historical failure rates. If the rate is deemed acceptable, this can be considered the societal target risk. A difficulty is to isolate failures truly related to code checks. Most structural failures occur because of blunders or gross errors in design concept, detailing or fabrication and are *not* related to the code checking. The approach usually adopted for establishing the target beta is to assess the present design provisions and perform safety index calculations over a wide range of representative practice (e.g., for different bridge spans, geometries, attachments). In general, there will be a wide range observed in computed β's because uncertainties were not consistently considered in the original development of the specifications. The aim in any new code provisions should be uniform or consistent target reliabilities. Thus, an average beta based on present standards is selected and this becomes the target for future code provisions.

2.3 Selection of Safety Factors

Figure 1 shows the beta vs. safety factor, γ, for a 100' span bridge and category C detail. The figure was obtained by substituting value a of γ in the expression for fatigue damage (Eq. 22 of Ref. 1) and computing the corresponding safety index, β using Eq. 1 herein. Plots similar to Figure 1 were made for different spans, and different fatigue detail attachments [5]. It was shown that the plot of β vs. γ is not sensitive to such changes so a single representative curve such as Figure 1 could be used. In the calibration, the target beta is selected as an average of the betas implicit in present design practice.

Fourteen AASHTO design cases (A-N) with different truck volumes, detail categories, spans impact factors, lateral distribution factors, and support conditions (simply supported or continuous) are shown in Table 1 to evaluate an average beta implicit in the present AASHTO design practice. This is done by taking sections which just satisfy the present AASHTO fatigue criteria and computing the implicit safety factor, γ. The designs selected were intended to be both representative of typical cases and also to represent possible extreme occurrences. For example, case H has a mean impact of 1.20 and a mean girder distribution of 0.50. The corresponding β's found from the γ versus β graph for each case are shown in Table 1. For example, for redundant or multiple load path members, Table 1 shows, that most of the design points fall around a beta of 2.0. This is midway in the range of the betas (0.7-3.6) corresponding to all the design points. The average β of 2.0 corresponds in Figure 1 to a reliability factor γ of 1.35. The same analysis was repeated for nonredundant details. The mean of the range of betas (1.5-5.3) for nonredundant details appears to be about 3.0. This corresponds to a reliability factor, γ, of 1.75. From this analysis of the average betas for existing design the target safety index for redundant and nonredundant members was fixed as 2.0 and 3.0, respectively, in the proposed evaluation procedure.

These examples demonstrate quite strongly the advantages of the proposed format. For redundant spans, we try and achieve our target β of 2 for all the design cases while AASHTO produced betas ranging from 0.7 to 3.6. Design with high betas is uneconomical, while the low betas will have relatively low probabilities that the actual fatigue life will exceed the predicted life. Similarly, for the non-redundant cases, the proposed evaluation methods try to achieve a target β of 3.0 for all cases compared to AASHTO betas that range from 1.5 to 5.3. The target betas will not be achieved exactly even for the proposed procedures. More factors would be needed to make this possible. The scatter in beta, however, will be smaller. The corresponding betas for these sections are shown in columns (5) and (7) of Table 1 for redundant and nonredundant members, respectively. The scatter in beta for redundant members is between 1.85 and 2.17 (as against 0.7 to 3.6 for AASHTO methods) and for nonredundant members it is between 2.85 and 3.10 (as against 1.5 to 5.3 for AASHTO methods). Hence, the proposed procedures achieve the goals of a more uniform safety index.

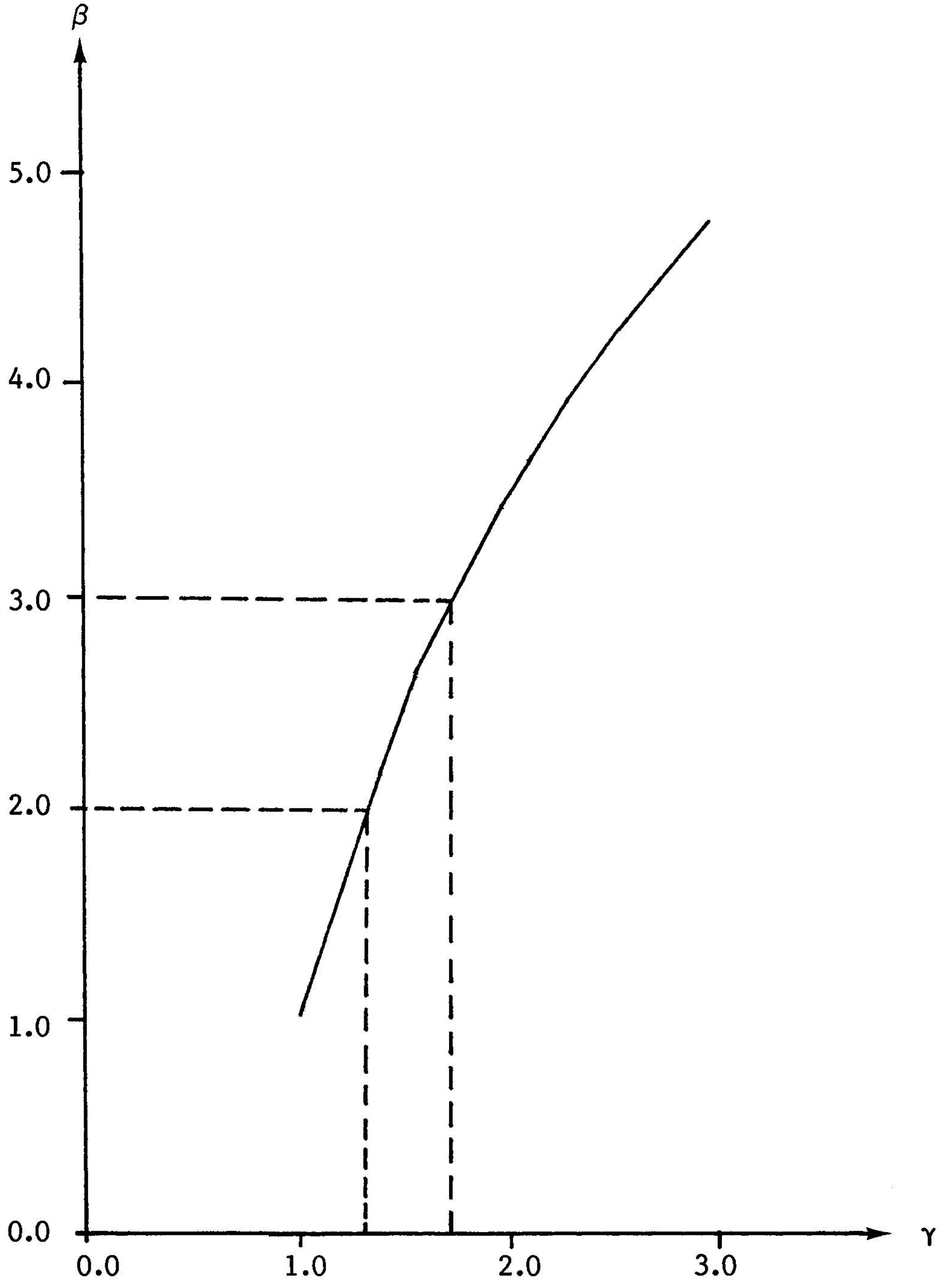


Figure 1. Mean γ vs. β curve.

2.4 Safety Factor Adjustments in Evaluation Procedure

The evaluation procedures have been developed with the following aims: (1) consistent and uniform reliability over the range of application (all spans, categories, lives, volumes); (2) flexible to incorporate site-specific data; (3) flexible to provide the Engineer with a better (usually longer) fatigue life estimate if more effort is applied. The first aim was satisfied by the calibration process. A target safety index has been fixed to achieve consistent and uniform reliability. The other two goals are obtained by incorporating several alternatives to the basic procedure. Some of the alternatives need site-specific data, while some other alternatives require more analysis effort by the Engineer. It should also be noted that in the evaluation, the Engineer can also obtain the mean fatigue life as well as the safe life. This should help explain why a span, which does not satisfy a safe life check, may not show any visual signs of fatigue cracking. Typically, the mean life will be about 5 times the computed safe life for redundant spans and 10 times the safe life for nonredundant spans. For a span not meeting the required safe life, the Engineer can select different options including more frequent inspection or control on vehicles.

Most of the alternatives in the evaluation lead to lower safety factors. The concept here is that risk is influenced by both the safety factor and the uncertainty. The same target safety index can therefore be achieved by reducing uncertainties and using a smaller safety factor. For example, availability of stress measurements for an attachment rather than the computed value permit the reliability factors, γ, to be reduced by 0.85. Use of measured truck weight spectra to provide a site-specific equivalent effective weight for the fatigue design vehicle reduces the γ's by 0.95. Finite element analysis to compute attachment stresses rather than the distribution chart or formulas provided in the specification reduce γ's by a factor 0.96.

2.5 Fatigue Limit

It is generally accepted that a constant amplitude fatigue limit exists for typical bridge details. If all cycles in a variable amplitude spectrum are below this constant amplitude fatigue limit, the fatigue life for the spectrum would be infinite. More research is still required to determine if a fatigue limit exists for a variable amplitude stress spectrum when only several stress cycles exceed the constant amplitude fatigue limit. Long life fatigue tests being conducted at Lehigh, TRRL, Maryland, and elsewhere may help resolve this issue. As one illustration,the reliability program was modified to exclude all the stress cycles below the constant amplitude fatigue limit while computing the fatigue damage. The results show that even after the model has been changed (a fatigue limit is introduced), the average betas from the proposed method and the AASHTO method are still almost the same [5].

3. PROPOSED FATIGUE EVALUATION PROCEDURES

A proposed bridge fatigue evaluation procedure based on the above reliability analysis was approved as a Guide Specification by AASHTO. Although optional as a requirement, it is being developed for a new edition of the commonly used AASHTO Maintenance Inspection Manual. This document is used to evaluate on a two year cycle, each of the nation's 600,000 bridges. The fatigue evaluation specification is primarily intended to determine the remaining safe life for application to inspection, maintenance, repair, replacement strategies, permit review and general truck weight control policies.

3.1 Application

This section summarizes the recently approved AASHTO Guide Specifications for fatigue evaluation of steel bridges [3]. The specifications for assessing remaining safe life apply only to uncracked members subjected to primary stresses that are normally calculated in design. The stress range is calculated first and used to predict the remaining mean and safe fatigue lives. The remaining mean life is the best possible estimate of the actual remaining life. The remaining safe life is a more conservative estimate which provide a level of reliability comparable to present AASHTO fatigue provisions. Alternatives that may be used at the option of the Engineer are given for several steps in the procedure. Each attachment must be considered individually.

The stress range for fatigue evaluation can be calculated by the following steps or, alternatively, can be determined from field measurements. A fatigue truck is used to represent the variety of trucks of different types and weights in the actual traffic. This truck has three axles with axle spacings of 14 ft and 30 ft and axle weights of 6, 24 and 24 kips respectively. This spacing approximates that for the 4- and 5-axle semitrailers that do most of the fatigue damage to bridges. The gross weight of the fatigue truck is 54 kip; this weight was developed from extensive weigh-in-motion data. Alternatively, the gross weight can be calculated from a truck-weight histogram obtained from weigh station or weigh-in-motion data for the site. Alternative axle spacings and weight distributions based on site data are also permitted.

The effects of more than one truck on the bridge at a time can be neglected unless there are conditions of close spacing of the trucks. The dynamic impact caused by different trucks usually vary considerably. Test data indicate that a factor of 10 percent is appropriate for fatigue evaluations. Poor joint or pavement conditions, however, require higher values.

The stress range is based on the moment or axial load range caused by the passage of the fatigue truck across the bridge. The lateral distribution factors for the fatigue evaluation are based on a single truck at the centerline of a traffic lane rather than on trucks in all lanes. Field tests have shown that the bending stress of actual bridge members is below that calculated by normal procedures, which conservatively neglect such effects as unintended composite action, contributions from nonstructural elements such as parapets, unintended partial end fixity at abutments, and direct transfer of load through the slab to the supports. To account for these effects, the computed section modulus is increased by appropriate percentages for composite and noncomposite sections, respectively.

Reliability factors are provided to assure adequate reliability in calculating the remaining safe life for different cases. Basic factors are recommended for redundant and nonredundant members. The corresponding probabilities that the actual remaining fatigue life will exceed the calculated remaining safe life are about 97.7 percent and 99.9 percent, respectively. The reliability factor is 1.0 for calculating the remaining mean life. If the compressive dead load stress is high enough so that essentially all of the stress cycles caused by normal traffic are completely in compression, the fatigue life is assumed to be infinite. If the maximum stress range in tension falls below the fatigue limit for a particular detail, crack growth will not occur and infinite fatigue life may be assumed. This situation, which applies primarily to higher detail categories (C and above), is checked by comparing the factored stress range with a fatigue limit value calibrated to a β of 2 to provide an adequate reliability that crack initiation will not occur. This fatigue limit value is equal to (1/2.75) times the present AASHTO allowable stress range for the over-two-million cycle category.

Finite remaining life: If a given detail does not satisfy the infinite life check, the remaining safe fatigue life (in years) corresponding to the factored stress range is calculated for a lifetime average truck volume and a selected number of stress cycles per truck passage. Alternatively, more refined procedures that involve growth rates and changes in truck weights with time can be used to calculate the remaining fatigue life. Detail constants are for the detail categories in the present AASHTO specifications. The equivalent number of stress cycles per truck passage is given for various cases.

Options if remaining life is inadequate: The evaluation procedure gives four options that may be pursued if the Engineer considers the calculated remaining safe fatigue life to be inadequate. These include (1) calculating fatigue life more accurately, (2) restricting traffic on the bridge, (3) repairing the bridge, or (4) instituting periodic inspections.

4. EVALUATION OF PROPOSED TRUCK WEIGHT REGULATIONS

In most industrialized countries there is increasing economic and political pressures to allow heavier vehicles and special permit trucks. The productivity gains are impressive from such weight increases and pavement damage can be mitigated or reduced by allowing vehicles with up to nine or more axles. A recent Bridge Impact Study by the writer was conducted for the Transportation Research Board under a congressionally mandated truck weight study [4]. It is only for bridges that increased truck weights may cause damages which offset the expected gains to freight shippers. Most of the bridge cost concerns strength capacity and increasing numbers of structurally deficient bridges. Fatigue costs were also considered. The approach used in the study is a reliability model similar to that given above for fatigue evaluation. This model gives the increasing risk of fatigue cracking as a bridge ages and accounts for different truck spectra. The probability that the actual fatigue life will be less than current life can be obtained from the reliability index.

4.1 Fatigue Damage

Fatigue damage is caused by all trucks passing over the bridge. If present truck weight regulations are changed, a percentage of the trucks in present traffic will be replaced by the new heavier trucks permitted. To calculate the relative fatigue damage caused by traffic before and after the change, it is necessary to know the composition of the truck traffic before and after the change; that is, the percentages of different types and weights of trucks in the traffic. In essence, the proposed fatigue cost model projects present expenditures for fatigue problems in steel bridges and adjusts these costs based on changes in traffic projections and passage of time.

4.2 Present Fatigue Cost

An estimate of the present expenditures on fatigue problems is required so that they can be used as a base case in projecting the expenditures associated with new truck weight scenarios. A limitation in a state survey is that agencies generally have great difficulty in assessing the costs of fatigue damage and relating these to vehicle regulations. In addition to the state survey, an estimate of the current annual cost of fatigue damage on steel structures in the United States was discussed with several experts on fatigue damage and repairs in highway bridges. For example, Dr. John Fisher stated that he has personal knowledge of fatigue problems on several hundred bridges. Based on these discussions an annual figure of $50 million would be an appropriate estimate. These include all costs to (a) repair actual or potential fatigue damage, (b) replace bridges that are unsafe because of fatigue damage and cannot be economically repaired, (c) perform engineering studies to evaluate actual or potential fatigue problems revealed by routine inspections or suggested by experience with similar bridges at other sites, (d) make engineering studies to develop appropriate corrective measures if required, (e) make extra inspections required to monitor fatigue cracking or check certain types of bridges that have had fatigue problems at other sites, and (f) reroute traffic when a bridge is closed because of fatigue problems.

Over the next 50 years, $50 million per year of damage (present value) totals $2.5 billion. There are 75,000 steel bridges on Interstate and primary routes and $2.5 billion in damage represents about 12% of the net value of such structures. This percentage is not unreasonable since present AASHTO fatigue design procedures are based on a 5% exceedance probability for fatigue life failure and further, many steel bridges were constructed before any fatigue provisions were in place and in recent years there has been significant increases in truck volume and average weight.

4.3 Future Fatigue Costs

The fatigue risk model is used to extrapolate the present fatigue damage costs as they increase in the future due to a) passage of time and accumulation of more cycles of loading, and b) changes in weight regulations which accelerate the damage rate. For different weight changes or scenarios the costs are found by computing fatigue cost vs. a damage ratio, R. The reference or base value for cost is $50 million per year. The damage ratio, R, is defined as:

$$R = \Sigma_i P_i R_i \tag{2}$$

where i - indicates span interval (including both simple and continuous spans)
P_i - percent of steel bridges in span interval i
R_i - applicable damage ratio for interval i.

$$R_i = \frac{\Sigma_t \Sigma_W V(t,W,S)\, M(i,t,W,S)^3}{\Sigma_t \Sigma_W V(t,W,B)\, M(i,t,W,B)^3} \tag{3}$$

where t - truck vehicle type
W - weight interval for truck type t
$V(t,W,S)$ - volume for truck type t, weight interval W, and proposed truck weight scenario S
$M(i,t,W,S)$ - maximum bending moment for span i, for weight W, and type t, in scenario S
$V(t,W,B)$ - base case (current) volume for type t, weight interval W
$M(i,t,W,B)$ - corresponding moments for base case

4.4 Future Damage

In the bridge evaluation, the reliability model is applied to a single bridge (with known design life and site parameters). The approach now is to extrapolate to the entire system of existing bridges. An average present life of 25 years was used as typical for existing Interstate and major route structures that are affected by fatigue problems. New weight regulations change the rate of "aging". A parameter y is used to express future damage, D_f, in terms of present damage, D_o.

$$D_f = D_o \frac{25 + (AGE - 25)\, R}{25} = D_o y \tag{4}$$

Where y = aging parameter
AGE = average bridge age at a future time compared to present 25 year age.
R = ratio of the annual damage after new weight regulations to present annual damage from Eq. (2).
D_o = damage accumulated at present
D_f = damage accumulated at future date

R is the ratio of damage before and after a change in truck weight regulations and accounts for both truck weight and volume changes. A productivity analysis was part of the truck weight study to project volume changes within each truck type and region in the U.S. Because of the wide range of design loadings and truck volumes for the population of existing bridges, the fatigue risk procedure must be calibrated for the average safety margin of the entire population. Based on the number of fatigue problems to date, a beta of 2.5 seems reasonable. This implies a probability of 0.006 for a fatigue failure on an existing bridge. A .006 risk would mean 450 fatigue failure. This is plausible in view of Professor John Fisher's comment above.

Using a β = 2.5, y = 1 (present age) and the risk model in Eq. (1) leads to a mean safety margin of 14. This means that the typical bridge has an expected life 14 times the present average life of 25 years. As the structure ages, and further damage is accumulated, beta decreases. For example, when beta falls to 1.5 in the future the risk for an average steel bridge will have increased from .006 (at the present time) to about 0.067. This represents an increased risk ratio of 11.2. Using the 75,000 bridge sample, instead of the cumulative 450 failed bridges at present, there is expected to be a cumulative total of 5,025 damaged bridges at that time. It is assumed that damage costs increase in direct proportion to the number of failed bridges. Thus, the total cost at the future time, when beta is 1.5, will be 11.2 times the present $50,000,000 or $560,000,000 per year. In order to compare with productivity benefits future damage costs are transformed to constant equivalent annual costs over the next 50 years.

4.5 Examples

Fifty years of future life are projected to cover the remaining life of the existing population of steel bridges. New steel bridges will be based on the new truck weight regulations to have adequate fatigue life and are not included in this cost analysis. For example, the equivalent uniform annual cost of fatigue damage with no change in present damage rate is $157M/year using i = 7%. If the new truck regulations double the damage ratio, R, to 2.0, the fatigue damage increased to $320M/year. Sensitivity studies were carried out to find the fatigue costs for different assumed present beta, interest rates and other statistical parameters [4].

4.6 Proposed Truck Weight Regulations

It is shown above that the present equivalent annual cost of bridge fatigue damage with no change in regulation is $157M/year. This is based on the fatigue risk model to extrapolate the present expenditures of $50M/year will increase as bridges "age" increasing the risk. Several different weight change scenarios were studied along with data from productivity projections to estimate the volume of truck changes. As a base case, the present U.S. legal truck weight limit is 80,000 lbs although many jurisdictions permit higher weight under special legal, legislative or administrative actions.

Adoption of a proposed Canadian Interprovincial truck regulation, for example, would increase the gross weight from 80,000 to 131,000 on a 65' wheel base as well as other weight increases for shorter wheel base vehicles. The fatigue damage increase using Eq. 2 was found to be R = 1.41 [4]. This raises fatigue damage costs by an estimated $65M/year. In general, it is found that truck weight regulations have a much greater cost impact due to causing strength deficiencies than shortening of fatigue life. Such conclusions assume that funds will be expended to increase strength capacity levels to requirements for increased truck weights. One possibility is that agencies will restrict heavy vehicles to special routes which do not require strength upgrades. In that case, the fatigue cost may be a significant proportion of the total bridge costs. In all, some 20 truck weight scenarios were studied using the FHWA data file of some 600,000 bridges. With respect to all weight regulations studied, productivity gains far outweighed bridge costs.

5. CONCLUSIONS

1. Reliability models are available to develop nominal fatigue checking formats and safety margins which lead to uniform and consistent reliability indices.
2. Sufficient data is now available to calibrate the model to use performance experience as a guide for computing target indices for singe and redundant load path systems.

3. A Guide Specification has been presented and adopted by AASHTO which is based on the model discussed herein and has been extensively used by several agencies.
4. Extensions of the fatigue risk model estimated future bridge damage costs if truck weight regulations permit heavier loadings and more rapid aging of structures.
5. It is important that increased tax revenue be available to the bridge system to ensure adequate funds for increased inspection, repair and eventually earlier replacement.

6. ACKNOWLEDGMENTS

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Table 1 Comparison of β's in proposed methods and present AASHTO methods

			β Redundant Members		β Nonredundant Members	
Designation	Span	Detail Category	AASHTO	Proposed	AASHTO	Proposed
(1)	(2)	(3)	(4)	(5)	(6)	(7)
A	120'SIMPLE	C	1.05	1.99	1.85	2.93
B	100'SIMPLE	C	2.00	2.02	2.90	2.96
C	60'SIMPLE	A	2.05	2.03	2.10	2.85
D	60'SIMPLE	B	2.15	1.97	2.65	2.92
E	60'SIMPLE	C	2.20	2.10	3.20	2.93
F	60'SIMPLE	D	2.75	1.97	3.90	2.91
G	60'SIMPLE	E	3.10	1.85	5.35	2.87
H	30'SIMPLE	C	0.70	2.16	1.50	3.09
I	100'CONT	A	1.45	2.06	1.50	2.89
J	100'CONT	B	1.60	2.01	2.00	2.98
K	100'CONT	C	1.65	2.05	2.80	2.99
L	100'CONT	D	2.05	2.01	3.45	2.97
M	100'CONT	E	2.40	1.90	4.80	2.96
N	60'CONT	C	3.55	2.17	4.10	3.10