

Reliability and Load Modeling for Bridge Management

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Managing the nation's bridge infrastructure requires major decisions on design and construction of new bridges, replacement, strengthening or posting of deficient bridges, issuing permits for truck overloads, and the implementation of truck weight regulations. These decisions have to be made after a careful review of the safety of the affected bridges. This paper illustrates how structural reliability theory can be used to provide tools for bridge management decisions. Using a reliability index as safety criteria, this paper describes reliability-based methods for the development of: a) Criteria for selection of load and resistance factors for a new LRFD bridge design code; b) Flexible load capacity evaluation or rating techniques for existing bridges; c) Fatigue evaluation procedures for steel bridges; And d) new bridge formulas for establishing truck weight regulations.

INTRODUCTION

Managing the nation's bridge infrastructure requires major decisions on design and construction of new bridges, replacement, strengthening or posting of deficient bridges, issuing permits for truck overloads, and the implementation of truck weight regulations. These decisions have to be made after a careful review of the safety of the affected bridges.

Insuring bridge safety requires a balance between the loads imposed on a bridge and the capacity of its members. Evaluating member capacity has traditionally relied on a visual inspection of the bridge while estimates of truck loads were obtained from the load envelope provided by AASHTO's HS or legal vehicles [1]. This traditional approach while very expedient ignores two fundamental issues: First, the current inspection process is very subjective and estimates of member capacities are associated with very high levels of uncertainty. Second, AASHTO's HS vehicles do not provide a consistent envelope over the entire range of bridge spans and geometries. Over the last few years, major efforts by the authors and other researchers have led to the development of new techniques for the evaluation of bridge safety using structural reliability theory [2,3,4,5,6,7,8].

Structural reliability methods contain the necessary ingredients for providing rational bridge management tools. Safety is expressed in terms of a measure of the probability that the capacity will exceed the load effect due to an extreme truck loading event. The maximum loading event is characterized by the number of trucks simultaneously on the bridge, their gross weights, axle spacings and the relative position of these vehicles. In addition, uncertainties associated with estimating the dynamic response and the distribution of loads to each bridge member are considered. A data base is used reflecting measured truck spectra, analysis variables, strength of materials, as well as projections of expected future loadings over bridge life spans. In this approach, design and evaluation factors are correlated to the

uncertainties inherent in estimating the strength of members, the behavior of structural systems and the loading of bridges. In addition, the consequences of a single member failure for redundant or nonredundant spans can be accounted for using this reliability technique.

This paper illustrates how structural reliability theory can be used to provide tools for bridge management decisions. The work is based on the results of recent studies by the authors for FHWA, TRB, NCHRP and Ohio DOT on load modeling and reliability analysis of highway bridges [3,4,6,7,8]. Using a reliability index as safety criteria, this paper describes reliability-based methods for the development of: a) Criteria for selection of load and resistance factors for a new LRFD bridge design code; b) Flexible load capacity evaluation or rating techniques for existing bridges; c) Fatigue evaluation procedures for steel bridges; And d) new truck weight formulas for establishing truck weight regulations. The latter application requires a review of the existing bridge inventory and allocation of resources to improve productivity while maintaining a consistently safe bridge system.

STRUCTURAL RELIABILITY THEORY

Load intensity, load effect analysis and structural strength parameters are not known with certainty. The aim of structural reliability theory is to account for the uncertainties in evaluating the load carrying capacity of structural systems or in the calibration of safety factors for structural design codes. Such uncertainties may be represented by random variables and their probability distributions.

The standard deviation of a random variable R with a mean \bar{R} is defined as σ_R . A nondimensional measure of the uncertainty is the coefficient of variation (COV) which is the ratio of standard deviation divided by the mean value.

Typical COV's for structural applications range from 8-15% for material strength, 5-10% for dead load, and 15-30% for live load and even higher for wind and seismic effects.

Codes often specify safe or nominal values for the variables used in the design equations. These nominal values are related to the means through bias values. The bias is defined as the ratio of the mean value to the nominal value used in design. For example, if R is the member resistance, R can be obtained from the nominal or design value R_n using a bias factor such that:

$$\bar{R} = b_r R_n \quad (1)$$

where: b_r is the resistance bias and R_n is the nominal value as specified by the design code. For example, A36 steel has a nominal design yield stress of 36 ksi but coupon tests show an actual average value close to 40 ksi. Hence the bias of the yield stress is 40/36 or 1.1.

In structural reliability, safety may be described as the situation where capacity (strength, resistance, fatigue life, etc.) exceeds demand (load, moment, stress ranges, etc.). Probability of failure, i.e., probability that capacity is less

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than applied load may be formally calculated; however, its accuracy depends upon detailed data on the probability distributions of loads and resistances. Such data is often not available so approximate models are used for calculation.

Let the reserve margin of safety of a bridge component be defined as, Z , where:

$$Z = R - S = R - (D + L) \quad (2)$$

R is the resistance or member capacity, S is the total load effect ($S=D+L$), D is the dead load effect, and L is the live load effect.

Risk is often described by a safety index (beta):

$$\beta = \frac{\bar{Z}}{\sigma_Z} \quad (3)$$

where \bar{Z} is the mean safety margin, σ_Z is the standard deviation of the safety margin. Probability of failure is the probability that the resistance R is less than the total applied load effect $P_f = \Pr [R < S]$

If R and S follow normal distributions then:

$$P_f = \Phi [-\beta] \quad (4)$$

Where Φ is the normal probability function that gives the relation between the probability that the normalized random variable exceeds a given value.

In general, beta's from either normal or lognormal models are used as estimates of the reliability of a structural member even if its capacity and applied load are neither normal nor lognormal. To improve on these estimates "Level II" methods have been developed [9]. Level II methods involve an iterative calculation to obtain an estimate to the failure probability. This is accomplished by approximating the failure surface ($Z=0$) by a tangent hyperplane at the point on the surface closest to the origin, after the surface is mapped into a normal space.

The safety index approach has been used by many code writing groups throughout the world to express structural risk. Betas in the range of 2-4 are usually specified for different structural applications. For new bridge constructions, specified target betas are relatively high, say on the order of 3.5. But, looking at the entire bridge life span, typical betas may fall to about 2.5. These usually correspond to the failure of a single component. If there is adequate redundancy, overall system reliability indices (betas) will be higher.

Calculation of beta is not made solely for making statistical risk statements but rather for recommending the proper load and strength safety factors for design or evaluation specifications. One commonly used approach is that each type of structures should have uniform or consistent reliability levels over all applications. For example similar beta values should be obtained for bridges of different span lengths, number of lanes, continuous spans, roadway categories, etc.

Appropriate target betas are obtained based on existing designs. That is, if the public is generally satisfied with the safety performance of bridges designed according to current criteria, then the safety index obtained from current designs is used as the target that any new design should satisfy. This calibration with past performance also helps to minimize any inadequacies in the data base. The calibration effort is usually executed by code groups as follows:

- Safety indices are calculated for current code design and performance of existing structures based on statistical information about the randomness of the strength of members and the statistics of applied loads. For medium to short span bridges, the load S in equation (2) is divided into two parts:

Dead load and live load. R on the other hand is determined by looking at the statistics of the resistance of typical bridge members. This is usually done for a range of applications such as different span lengths, girder spacings, material types and traffic conditions

- In general, there will be considerable scatter in such computed safety indices. If the existing code is believed to provide an average satisfactory performance then, a target beta can be directly extracted. This is done by examining the performance and experience with selected bridge examples and averaging the beta's

- Safety factors and nominal loads and strengths for a new format are selected by trial and error to satisfy the target beta as closely as possible for the whole range of applications.

DATA BASE AND LIVE LOAD MODELS

To execute the safety index calculations, one needs to obtain the statistical data of all the random variables that affect the safety margin Z of equation 2. These are the member resistances, the dead load effect and the live load effect. Experimental and simulation studies have developed estimates for the statistics of member resistances for different types of bridges. Data on the live load statistics are however less common and in fact, besides the limited data from the Weigh-In-Motion studies at Case Western Reserve University, little information is available on bridge related truck load statistics in the United States [16]. This section presents a summary of the statistical data used by the authors and their colleagues in several studies on bridge reliability.

Dead Load

Dead load effects are obtained from the self-weight of the structure accounting for the weight of asphalt and other non-structural members. Hansell and Viest [10] found that the nominal dead load effect for steel members is related to the design live load and the span length by the formula

$$D_n = 0.0132 (L_n + I_n) SL \quad (5)$$

where L_n is AASHTO's design live load effect on a member. I_n is the nominal impact load effect on the member. SL is the span length in feet. The mean dead load value D was also obtained from equation (5) i.e. the dead load bias is estimated at 1.0. The dead load coefficient of variation used is 9% based on the typical values given in ref [11]. The same relationship between the dead load and span length was used for prestressed concrete bridge members based on the data provided by James et al [12]. Imbsen et al. [13] recommended a similar formula for concrete T beams such that

$$D_n = (L_n + I_n)(0.6967 - 0.00762SL + 0.0002554SL^2) \quad (6)$$

Resistance Data

Summarizing earlier work by Galambos [14] and Ellingwood [15], Moses and Verma [3] established biases and COV for different categories of steel members and prestressed concrete members. For example, steel members in new

condition were assigned a bias of 1.1 relative to the nominal capacity as specified by AASHTO procedures and a COV of 12%. Partially corroded steel members with some slight loss of section were associated with a bias of 1.05 and a COV of 16%. Severely corroded sections with noticeable loss of section have a bias of 1.0 and a COV of 20%. For prestressed concrete members in good condition a bias of 1.05 and a COV of 9% were used. These biases and coefficients of variation account for the uncertainties in the material properties, fabrication and scatter in prediction theory.

Live Load Modeling

Bridges are designed to safely withstand the maximum load expected over the service lifetime of the structure. In short to medium span bridges, maximum live load is usually due to the occurrence of several heavy trucks simultaneously on the bridge. Each occurrence of one or more vehicles on the bridge (herein called loading event) is characterized by the number of trucks in the event, their gross weights, axle spacings, axle weight distribution and the relative position of these trucks with respect to each other. All these factors are random variables which should be accounted for in a model to calculate the maximum loading on a bridge.

Simulation programs have been developed to study the truck loading problem [6]. In these programs, the bridge is divided into slots and a truck loading event occurs when there is at least one truck on any one of the assumed slots. Each truck involved in the loading event was assumed to be either of the single or the semi-trailer type. Each truck in the event will also have a different gross weight. Depending on the truck type, each truck involved in the event will be associated with a gross weight and a corresponding probability obtained from gross weight histograms for the different truck types considered.

Given the truck positions and given the gross weights of all the trucks in the event, the maximum moment response associated with the event can be easily calculated from the influence line of the bridge. The response of the bridge due to the event is also associated with a headway probability and probabilities of the gross weights of the trucks. The corresponding moment response is then associated with a probability equal to the product of the headway probability and the gross weight probabilities. This assumes independence between the headway and the gross weights and between the gross weights of the different trucks in the event.

Based on the results of the simulation program, the median of the total response of the maximum load in 50 years for a general truck traffic at a given site is approximated herein by the load formula [6]:

$$M = a m W_{.95} H \quad (7)$$

where a is a deterministic value dependent on the standard truck configuration used in the simulation, the span length and the response variable (midspan moment, end shear ...). m is a random variable reflecting the type of truck traffic configuration present at the site e.g. single, semis, etc. It is also a function of span length. H is a random variable and gives the overload factor due to the presence of closely spaced vehicles, side-by-side and following vehicles. H also reflects the probability that vehicle weights exceed the 95th percentile in combination with closely spaced events. It was found from the simulation model as discussed in the previous

paragraph and in reference [6]. H is a function of the truck volume and depends on the span length. $W_{.95}$ is a 95th percentile characteristic value of the truck gross weights.

Equation 7 is used in estimating the maximum live load applied on a bridge structure in its lifetime (usually taken as 50 years). This equation gives the total static load on a bridge. To obtain the load on a member under highway traffic, two additional factors are required and these are the impact factor (or dynamic amplification factor) i , and the girder distribution factor, g . The total load effect on a bridge member L is then the product of the maximum lifetime static load effect, the girder distribution factor and the dynamic amplification factor:

$$L = a m W_{.95} H g i \quad (8)$$

For a fifty year design, possible growth in the weights of heavy trucks traveling over the highways should be included in the reliability analysis. The approach adopted here is to include load growth explicitly as one of the variables denoted as Gr . A mean Gr factor of 1.15 along with a C.O.V. of 10% were assumed for the evaluation of the safety indices. The live load formula used in the safety index calculations for new designs becomes:

$$L = a m W_{.95} H g i Gr \quad (9)$$

Except for the factor a , all the variables of Eq. 9 are random variables with statistics based on examination of a number of sites as given in Table 1 for a fifty-year projection of the maximum load effect based on Weigh-In-Motion data collected at several sites [6].

Statistical data based on field measurements and theoretical analysis were collected by Moses and Verma on the girder distribution factor g . They found that for steel bridges a bias of 0.90 with a COV of 13% exists between the AASHTO recommended girder distribution factor and the values obtained by researchers. For concrete T beams, the bias obtained was 1.05 with a COV of 13%. Prestressed concrete bridges were associated with a bias of 0.96 and a COV of 8%..

Similarly, the impact factor was found to be a function of surface roughness. Three different values were recommended for the mean dynamic impact, these are 1.1, 1.2 and 1.3 for smooth, medium and rough surfaces respectively, these were all associated with a COV of 10%.

Two different $W_{.95}$ values are used depending on whether the span length is dominated by the loading of single unit trucks (spans less than 60 ft.) or dominated by the semi-trailer load (greater than 60 ft.). These are found to be 47 kips and 75 kips respectively.

The data used for new designs has to be averaged from several sites because no specific information is available on the type of trucks or the loading and response of the bridge before its construction and opening to traffic. Therefore, different statistics are used for the evaluation of existing bridges since site-specific data can be collected for those. Also, projections of maximum loads over fifty years are needed for the assessment of the risk over the design life span of the bridge. Shorter projection periods (two years) were recommended by Moses and Verma [3] for the evaluation of the safety of existing bridges. The two-year period was chosen because it corresponds to the bridge inspection period mandated by FHWA.

Moses and Verma also observed that truck traffic data can be divided into four categories which correspond to combinations of light truck traffic or heavy truck traffic on sites that have high truck weight enforcement levels or

unenforced sites. Different H and W values are recommended for each type of traffic [3].

BRIDGE CODE CALIBRATION

The first and most direct application of structural reliability theory and the bridge load modeling procedure outlined above is in the calibration of a new bridge design code. As stated above, development of a new code requires first the determination of a target safety index. Since engineers are generally satisfied with the performance of bridges designed with the current standards, Ghosh and Moses [7] decided to use the average safety index inherent in the current design practice as the target safety index that the new steel design code should achieve uniformly for all design applications.

Determination of Target Beta

The resistance R of a member designed using the current code has a nominal value that can be calculated from either the WSD or the LFD formulas. In both cases, the nominal load is obtained from the AASHTO design vehicle.

The calculations of the safety indices inherent in the existing code were executed assuming that all the random variables follow lognormal type distributions, the failure function used becomes of the form:

$$Z = R - D - a m W_{.95} H g i Gr \quad (10)$$

The results of the safety index calculations are given in Figure 1 for the two design cases considered. As can be seen, the safety index beta for all of AASHTO's design possibilities is inconsistent from span to span. For example the WSD method gives a maximum of 4.33 for the 200 ft span while the minimum is 2.38 at 30 ft. For LFD, the maximum index is 3.70 at 60 ft and the minimum value is 2.56 at 30 ft. These inconsistencies in betas suggest two needed changes in the current design: 1) changes in the nominal design load formula to give safety indices compatible with the actual loadings on the members and 2) revision of the load factors to provide more uniform betas. The target beta was chosen here to be 3.5 based on the average index from AASHTO's WSD and LFD which have a combined average from all span lengths considered of 3.46. This same target value was also used by Nowak [17] in the calibration of Ontario Highway Bridge Code and is currently being used by Kulicki [5] for the calibration of the new AASHTO design code.

Proposed Strength Design

In order to obtain more uniform beta's over the range of span lengths considered, a new nominal design load was recommended [7]. This new design model consists of using different vehicle types with corresponding truck gross weights to calculate the maximum design load effect on a bridge structure. In this respect, it is closer to the load model contained in the AASHTO rating manual [18]. The partial safety factors associated with the proposed loading shown in Figure 2 are : a resistance factor of 0.9, a dead load factor of 1.3 and a live load factor of 2.35. These factors were obtained by a trial and error selection to achieve the target safety index of 3.5 as closely as possible for all span lengths. The proposed design checking equation becomes:

$$0.9 R_n = 1.30 D_n + 2.35 (L'_n + I'_n) \quad (11)$$

R_n is the nominal girder strength, D_n is the calculated dead load effect, L'_n is the static load due to the maximum effect from the three design vehicles shown in Figure 2. These vehicle configurations and axle loads were also selected by trial and error to achieve uniform betas over the range of span lengths considered (30-200 ft). Notice that the first two vehicles of Figure 2 are similar to the single unit and semi-trailer trucks most commonly observed on U.S. highways. The third vehicle in Fig. 2 is similar to double trailer type trucks and its moment effect replaces AASHTO's Lane Loading for simple spans up to 200 ft. Calculation of I'_n follows the same impact calculation contained in AASHTO. The load effect on a member is obtained using AASHTO's distribution factors on the wheel load, (such as $S'/5.5$ for steel girders where, S' is the maximum spacing between girders in feet).

The safety indices obtained from the new checking equation are very uniform when compared to the existing code [7]. For example, the maximum beta is 3.60 for a 150 ft span and the minimum value is 3.44 for the 200 ft span. These small differences between the maximum and minimum beta values confirm that the goal of more uniform betas was achieved from the proposed new design format including the changes in nominal load model and the load factors.

LOAD CAPACITY EVALUATION OF EXISTING BRIDGES

In a study for NCHRP, Moses and Verma [3] also proposed to use reliability theory and the load modeling techniques presented above to develop load capacity evaluation procedures for existing bridges. The new rating equations proposed by Moses and Verma have the following format:

$$\phi R_n = \gamma_D D_n + \gamma_L (L_n + I_n) R.F. \quad (12)$$

where ϕ is the resistance factor, γ_D is the dead load factor and γ_L the live load factor. R.F. is the rating factor. Therefore, bridges which are just safe, according to this equation, have a R.F. equal to 1.0.

Determination of Target Beta

Establishing the target safety index was again an important step in the calibration process. In evaluation, there are at least four different procedures for rating currently in use in different States. These include the working stress WSR operating (WSR-O) and inventory levels (WSR-I) and the load factor LFR operating (LFR-O) and inventory (LFR-I) levels. Also, States may apply different distribution factors or nominal loadings (e.g. AASHTO legal vehicles, HS20 or other vehicles).

Figure 3 compares the betas for a set of hypothetical simple spans which just satisfy a R.F. = 1 based on WSR-I using present procedures. The live load loading category assumed is unenforced, heavy traffic and the nominal loading is taken to be the HS-20 design vehicle. The betas are compared to spans with a R.F. = 1 using LFR-I. It can be seen that the WSR-I betas are in the range of 3.8 to 4.6 for most spans while LFR-I betas vary between 3.0 and 4.6. The target beta (based on past practice) therefore depends on what criteria is used. Calculations for all the four traffic categories considered

show a large variation in the safety indices, depending on the severity of traffic (and hence the projected live load effect). The target beta values were selected based on the operating LFR for situations of enforced heavy traffic criteria. Using this approach the resistance and live load factors were calibrated to achieve a beta of 2.3.

For good condition nonredundant elements, the target safety index was derived from current inventory levels and an average value of 3.5 was chosen. It should be noted that the calibration was done using different levels of traffic for redundant and non-redundant elements. This is because calibration to the original design or inventory level should account for the severest traffic possible (which corresponds to unenforced, heavy traffic) whereas calibration to the operating level should be for enforced, heavy traffic ("reasonably controlled" according to the AASHTO Manual).

Calibration of Resistance and Load Factors

Based on the calibration procedure, a value of $\phi = 0.95$ was selected as the reference case for a good condition element for both steel and prestressed concrete. The value of ϕ is subsequently adjusted to recognize the data available from inspection and the degree to which maintenance can be expected to detect any important defects. For example, a member that shows a high amount of corrosion will have larger uncertainty in strength. This will produce a set of resistance factors as shown in Table 2. Thus, nonredundant components which require higher betas must be assigned a smaller ϕ to provide the higher reliability. γ_D was selected to be 1.20.

Because they provide generally uniform reliability levels, and because they also are familiar to engineers, AASHTO's rating vehicles were used as the reference nominal loadings in the new rating formulation. In addition, a lane loading was developed to cover the long and continuous spans (Figure 4).

Different load factors are used to realize constant reliability for each of the truck traffic cases presented above. For example, the selection of γ_L for each case is illustrated in Table 3 to produce the same uniform beta value over all spans. Thus, sites with the heavier truck traffic will require a higher load factor.

In addition, the proposed format gives corrections on the resistance factors and the live load factors depending on the level of maintenance and inspection, the use of advanced methods or field measurements to estimate the girder distribution factors and the number of lanes of the bridge.

DESIGN AND EVALUATION FOR FATIGUE DAMAGE

Moses, Schilling and Raju [4] also developed a similar approach for the fatigue design and evaluation of steel members. The nature of the fatigue process however, required a modification of the failure function. The safety margin was then defined as:

$$Z = Y_F - Y_S \quad (13)$$

where failure occurs if $Z < 0$. Y_F is life at which failure actually occurs and Y_S is the specified or calculated deterministic life. A linear Miner cumulative damage rule was assumed such that failure occurs when the

nondimensional damage accumulation sum equals 1.0.

A new fatigue design and evaluation equation was then proposed involving a safety factor γ . Different safety factors γ achieve different safety index values (beta's). The safety factor to be selected is sensitive to the target beta and hence to the statistical parameters of the variables in the reliability expression. During the calibration process, the target beta was selected as an average of the beta's implicit in present design practice. Fourteen AASHTO design cases with different truck volumes, detail categories, spans, impact factors, lateral distribution factors and support conditions (simply supported or continuous) were used to evaluate an average beta implicit in the present AASHTO design practice. This is done by taking sections which just satisfy the AASHTO fatigue criteria and computing the implicit safety factor, β , by the methods described above. It was found that most of the design points fall around a beta of 2.0, where the range of betas is between 0.7 to 3.6. The target beta 2.0 corresponds to a safety factor, γ , of 1.35. The same analysis is repeated for nonredundant details and the results showed that the mean of the range of betas for nonredundant details appears to be about 3.0. This corresponds to a safety factor, γ of 1.75. From this analysis the target safety index for redundant and nonredundant members was fixed at 2.0 and 3.0 respectively, producing safety factors of 1.35 and 1.75. A fatigue design vehicle with a 54 kip equivalent weight along with a new method to select the girder distribution factor and the design stress range were also proposed to produce more uniform safety levels than currently observed [3].

DEVELOPMENT OF NEW TRUCK WEIGHT FORMULA

The same reliability and load modeling techniques were also used by Ghosn [8] to develop a new truck weight (bridge) formula to regulate maximum weights of trucks over federal interstate highways. The load modeling equation (eq. 9) was used to develop the new legal weight limit formula. The proposed procedure accounts for the truck load effect by considering the statistics of current truck gross weights, multiple occurrence on the bridge and truck configuration. Also indirectly considered in the H factor is the truck traffic volume. It is also assumed that changes in truck volume and other regulation changes over the service lifetime of the bridge might produce an average increase in the live load effect of 15% ($G_r = 1.15$) the COV associated with this random variable is 10%.

Calculation of Required Load Effect Envelope

A target beta was also extracted from current design and loading as executed in the development of new design procedures. It was decided to use a target index equal to 2.5 to calibrate the new bridge formula. This value is similar to the average safety index for operating stresses which are regularly used by a number of states for rating existing bridges.

During the execution of the calculations, it was assumed that changes in the legal weight limits will produce a shift in $W_{.95}$ such that the ratio of $W_{.95}$ to the legal limit remains constant. The live load envelope required to produce an acceptable safety level is calculated as follows: Assuming that

current bridges are designed according to AASHTO's WSD method using HS 20 vehicles, the new mean live load \bar{L} is calculated such that the target reliability of 2.5 is exactly matched for each span length considered.

Calculation of Truck Weight Formula

Using the load envelope developed, a bridge formula was obtained using the procedure outlined by the TTI study [12]. The bridge formula was designed to give a relationship between the weight of a truck and its length. The steps involved are summarized as follows:

- A truck satisfying the bridge formula was assumed to have a total weight W_t and total truck length B.
- Assume the truck weight W_t is uniformly distributed over the truck length B.
- Several values of B are used such that B varies between 1 and 120 ft.
- Given a truck length B, find the moment effect M_r of a unit load uniformly distributed over B. This is done for each span length.
- For every span length, find the load envelope M_t required to obtain the target safety index 2.5.
- For each span length, find $W_{.95t}$ (target weight) such that $mM_r W_{.95t} H$ is equal to M_t . a is not included herein since it is implicitly considered in M_r .
- Repeat the previous steps for every truck length B. For each truck length, several $W_{.95t}$ values are obtained corresponding to the load effect of every span considered.
- For each truck length B find the lowest $W_{.95t}$ value: $W_{.95t \min}$.
- A bias factor gives the relationship between the legal load W_t and $W_{.95t}$. A bias of 1.07 is used for spans greater than 60 ft based on current truck weight statistics for semi-trailer trucks. The bias factor for shorter spans governed by single unit trucks is 1.09.
- Plot $W_{t \min}$ versus B. This curve provides an envelope that the distributed load W_t should satisfy in order to insure that all span lengths will produce a safety index of at least 2.5.
- Find an algebraic expression that will fit the $W_{t \min}$ versus B curve as close as possible. The proposed truck weight (bridge) formula obtained by fitting the $W_{t \min}$ vs. B results is as follows:

$$\begin{aligned} W &= (1.64 L + 30) 1000 & \text{for } L < 50 \text{ ft} \\ W &= (0.80 L + 72) 1000 & \text{for } L > 50 \text{ ft} \end{aligned} \quad (14)$$

Cost Analysis

A major portion of the total cost impacts for new regulations is the effect on the existing bridge population. Some 130,000 bridges are now rated structurally deficient with an estimated \$53 billion replacement or upgrading cost. If a new truck weight regulation introduced higher legal loads, a larger number of structural deficiencies will increase replacement costs. The procedure outlined herein for the cost analysis, follows the method developed by Moses for TRB's

Truck Weight Study [19].

To provide a base case for the cost analysis, bridges under current truck regulations are checked by vehicles corresponding to the current Federal Bridge Formula. This means that they should provide adequate capacity under the AASHTO legal vehicles. A bridge is considered deficient under this base case scenario if these AASHTO vehicles cause stresses that exceed the operating stress level plus 5% tolerance. The operating stress level is obtained based on a safety factor 1/0.75 and a load L_n obtained from the AASHTO rating vehicles not the HS loading. These proposed criteria are similar to the rating methods used by many state agencies.

The cost allocation study considered a large sample of bridges that represent the highway classifications and regions in the U.S. Bridges of different spans, geometries, material and age were analyzed. The sample was obtained from the Federal Highway National Bridge Inventory System (NBI). The predictions using the Base Case model produced a total number of deficiencies close to the 130,000 estimated deficiencies which appears in the Secretary of Transportation annual report.

The current US bridge network consists of 600,000 bridges some of which would need upgrading if a new legislation allowing higher truck weights is implemented. In this analysis, it is assumed that all deficient bridges under any proposed legislation will have to be replaced. The possibility of posting or closing bridges has been ignored because these options will entail economic and productivity costs to the shipping industry exceeding the cost of replacing the affected structures. Upgrading options are seldom used in practice because of Federal rules requiring that upgraded bridges should satisfy all regulations on geometry, lane widths, side barriers... Thus the cost estimates given herein are upper bounds to the values that will actually be encountered.

As previously mentioned, additional deficiencies are defined with respect to operating stress levels plus 5% load tolerance using legal vehicles satisfying the bridge formula under consideration. Replacement cost of deficient bridges do not reflect the existing condition and age of the structures. Table 4 shows the estimates of current number of deficient bridges in comparison to the expected number under the new regulations given in eq. (14). This is presented for two different highway categories: primary systems and non-primary systems. The estimate is also given separately for steel, reinforced concrete and prestressed concrete bridges. The cost implications of such number of deficiencies is given in terms of total length of deficient bridges. Construction costs are usually given for unit area and since bridges have standard widths, the total length of deficient bridges is directly related to the total cost.

The results of Table 4 show that a large increase in the total length of deficient steel bridges on primary highway systems will accompany the implementation of equation (14). The increase in the length of deficiencies is about 5.7 times the current length. The additional length of deficiencies for the other types of bridges and highway classification is in general less than 2.9 the current levels.

One reason for this large number of deficiencies is that many existing bridges do not satisfy the WSD HS-20 criteria used to determine the target reliability level.

CONCLUSIONS

A review of recent work on the application of structural reliability theory and load modeling for the development of bridge evaluation methods has been presented. Bridge management requires the consideration of the structural safety of the bridge and the safety of its users. The techniques presented above illustrate how safety criteria can be included while decisions on bridge design, closing or rating are done or while determining the load capacity of existing bridges in order to allow higher legal limits or permit loads.

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TABLE 1. Input Data for Reliability Analysis

Span (ft.)	a	m		H	
		mean	C.O.V.	mean	C.O.V.
30	6.07	0.92	15%	2.63	10%
40	8.57	0.93	12%	2.69	10%
60	13.57	0.94	6%	2.75	10%
80	13.40	0.93	9%	2.78	7%
100	18.40	0.95	7%	2.80	7%
125	24.40	0.96	6%	2.86	7%
150	30.90	0.96	5%	2.87	7%
175	36.90	0.97	4%	2.98	7%
200	43.40	0.97	4%	3.05	7%

Table 2. Resistance Factors for Different Member Categories

Resistance factors ϕ		Comment
nonredundant	redundant	
0.80	0.95	good condition
0.70	0.85	slightly corroded
0.60	0.75	heavy corrosion

Table 3. Live Load Factors for Bridge Evaluation

Category	Factor
- Low volume with reasonable enforcement and control of overloads	1.3
- Heavy volume with reasonable enforcement and control of overloads	1.45
- Low volume with significant overloads	1.65
- Heavy volume with significant overloads	1.80

TABLE 4. Consequences of Implementation of Proposed Bridge Formula on Simple Span Bridges

	Total # of Bridges	Current Deficiencies		Increase in Expected Deficiencies	
		Number ft	Lengthx10 ³	Number ft	Length.x10 ³
Primary Highways					
Steel	41140	1738	119.3	7248	679.5
R/C	25510	1174	43.7	2346	124.4
P/C	26474	1264	95.6	2896	249.7
Non-primary Highways					
Steel	154401	80243	3759.1	18742	1234.5
R/C	81671	21510	652.8	11957	521.0
P/C	48568	5503	247.5	6079	378.9

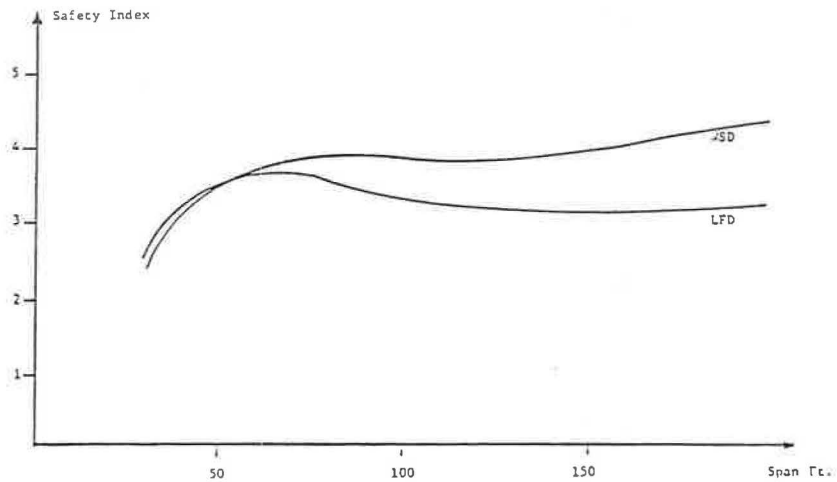


Figure 1 - Safety Indices with Current AASHTO

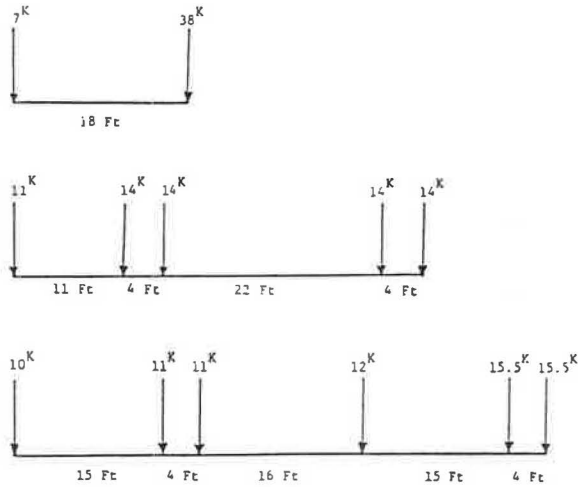


Figure 2 - Proposed Design Vehicles

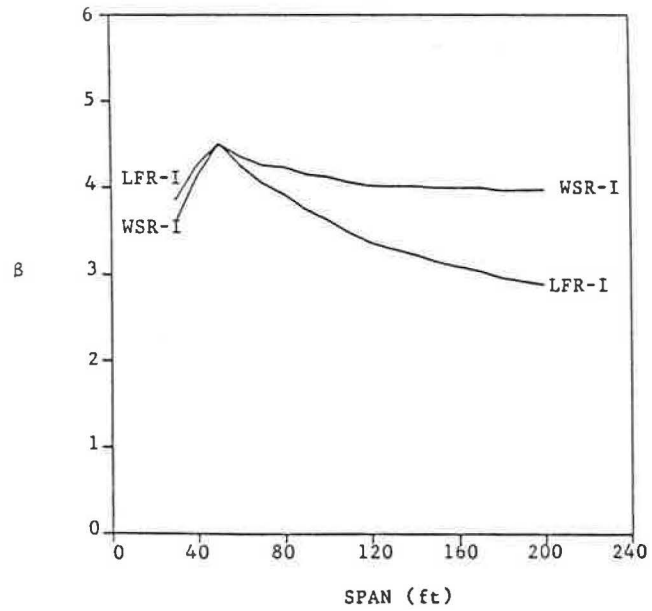
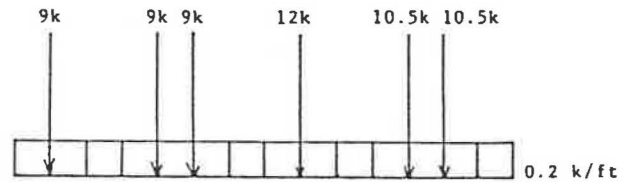


Fig. 3 - β for inventory checks (unenforced, heavy traffic)



- Lane loading

Figure 4 - Proposed lane loading for Load Capacity Evaluation