# RELIABILITY APPROACHES TO BRIDGE SAFETY AND TRUCK LOADING UNCERTAINTIES

Fred Moses, Case Western Reserve University

Design decisions for highway structures can utilize probability and statistics to express uncertainties in vehicle loading, analysis, strength and construction control. Structural reliability research is described to provide design codes with consistent risk levels and optimal designs. A detailed reliability approach is presented for deriving load and performance factors for steel element fatigue design. The uncertainties in truck weight, volume, headway, strength distribution (analysis), impact and fatigue life are included. The fatigue load model is extended to strength design by considering two behavior levels. The first level utilizes a limit state format with element-oriented load and performance factors derived for components with failure criteria such as maximum moment. Ultimate strength is recognized in a second level check with system coefficients based on the ratio of the load causing significant bridge distress to the limit state load. Code oriented research is described to derive system coefficients for various types of bridge structures using nonlinear and ultimate load analysis. The goal is to utilize the load margin between an element limit state and major bridge damage to contain load uncertainties in future load growth and overweight vehicle operations. The sparce load data available has inhibited introduction of reliability oriented specifications. A project is described for undetected weighing of vehicles in motion using instrumented highway bridge girders. The field results show its feasibility and opportunities for filling in missing data on load history and overweight vehicles.

The design decisions for highway structures involve uncertainties in loading, analysis, strength and construction control. Historical experience is an excellent guide for evaluating proposed design changes which affect the safety margin. This experience should also be supplemented by probabilistic and statistical analysis which quantifies these uncertainties.

The codified variables for controlling designs are the prescribed safety factors. These factors should reflect the degree of uncertainty in predicting behavior and the risk and consequences of not meeting performance criteria. Recent research in structural reliability theory has shown how load, analysis and strength uncertainties can be quantified to determine their impact on rational safety factors (1). In these studies, uncertainties are described as random variables and through probabilistic analysis the risk of damage or unserviceability is computed. This computation is often approximate to reflect the limited data base in most structural situations. Design factors can still be derived, however, to achieve structures with more uniform risk levels based on the corresponding uncertainties.

Code developments using structural reliability theories have been studied for adoption in steel buildings, concrete structures and offshore platforms (2,3,4). These reliability changes can also affect the economy of the structure. For example, savings in bridge structures would result by reducing design uncertainties through more load-data collection, accurate behavior analyses or improved construction quality control.

There are some important differences between highway bridges and buildings in applying structural reliability principles. Buildings are affected primarily by environmental (wind and seismic) loads which can be analyzed statistically from historical data. In bridges, especially short and medium span structures, the major loading is due to heavy trucks. Its statistical description is not precisely known due to limited and possible biased data and furthermore, evolves over time after the bridge is constructed. In addition, truck loads are repeated by millions of passages during a bridge lifetime so the statistical problem of estimating maximum loads (which also include multiple presence of more than one vehicle on the bridge) is quite formidable.

A reliability oriented strategy requires load statistics and design factors to provide safe structures, i.e., meet strength and serviceability limits and also be economical over long periods of time. An example of a code "strategy" is the AASHTO specification of its design vehicle (5). This vehicle weighs less than the legal load limit and is also considerably lighter than vehicles known to use the roadways, either by permit or illegal passage. Safety is maintained, however, by conservative distribution and material safety factors which are more than adequate based on load tests and examples of

bridge performance (6,7,8).

A number of studies have investigated the problem of bridge live loads and safety (9,10,11). A recent Symposium brought to light practices from different countries (12). The need for a rational methodology to derive design loads also became apparent in research for a limit state oriented bridge code (13). Similar problems in assessing load and bridge behavior uncertainties affect the normal operation of authorizing permit vehicles and establishing procedures for rating of existing bridges (14).

This paper presents a reliability-based framework for bridge design and analysis, especially the rational determination of truck loadings. The intent is not to present detailed code recommendations but rather to explore a general methodology for deriving reliability oriented bridge codes. The first problem described is a loading model to predict fatigue behavior. The uncertainties in truck weight, headway, volume, stringer distribution (analysis), impact and fatigue life are presented. A design calibration is described to produce uniform but small probabilities of premature cracks. An extension of this loading model is described for strength design and a strategy is described for achieving a uniform reliability by also considering the ultimate behavior of the bridge. A major factor in a bridge analysis is accurate load data because of the frequency of overloaded vehicles on our various highway systems. The paper describes a current project at Case to obtain such data by using highway bridges as load scales to weigh, undetected, vehicles in motion. Possible applications of the system are described

## Reliability Theory

The fundamental problem of structural reliability is illustrated in Figure 1. A single element with strength, R, is subjected to a load, S. R and S represent the random variables associated with strength capacity and loading including analysis uncertainties. These random variables may be described by frequency distributions. The overlapped portions of the curves in Figure 1 indicated regions of risk where load (S) exceeds capacity (R). The probability of failure ($P_f$) or risk may be calculated from these frequency distributions ($f_R(r)$ and $f_S(s)$) as

$$\text{Risk, } P_f = \text{Probability } [R < S]$$

$$= \int_S [\int_o^s f_R(r)dr]\ f_S(s)ds \qquad (1)$$

Risk decreases with increasing safety factor (initial cost). We can express the total cost ($C_T$) as illustrated in Figure 2 as

$$C_T = C_I + (P_f)C_f \qquad (2)$$

where:
$C_T$ = total cost
$C_I$ = initial cost
$C_f$ = cost of failure

An optimum or minimum cost occurs when the slope of initial cost ($C_I$) equals the negative slope of equivalent failure or insurance cost ($P_f \times C_f$).

The calculations in equations 1 and 2 can be used with precision only if there is high confidence in the load and resistance statistical data. Otherwise, relative measures of risk have been proposed which contain the important reliability parameters and can be used in deriving code safety factors (15,16,17). One simple approach is to define a safety margin (Z).

$$Z = R - S \qquad (3)$$

Failure occurs if Z is negative. The most important statistical parameters of Z are its mean ($\bar{Z}$) and variance ($\sigma_Z^2$) expressed as

$$\bar{Z} = \bar{R} - \bar{S} \qquad (4)$$

$$\text{and } \sigma_Z^2 = \sigma_R^2 + \sigma_S^2 \qquad (5)$$

A measure of risk is the safety index ($\beta$) or the number of positive standard deviations contained in the safety margin. (If R and S are normal variables, entering $\beta$ in the standard normal distribution table gives the risk.) To arrange a design format, note that (17)

$$\beta = \frac{\bar{Z}}{\sigma_Z} = \frac{\bar{R} - \bar{S}}{\sqrt{\sigma_R^2+\sigma_S^2}} \qquad (6)$$

$$\text{and } \sqrt{\sigma_R^2+\sigma_S^2} \simeq 0.7\ (\sigma_R + \sigma_S) \qquad (7)$$

Making the latter substitution in equation 6 and rearranging terms gives a safety check as

$$(1 - 0.7\beta V_R)\ \bar{R} \geq (1 + 0.7\beta V_S)\ \bar{S} \qquad (8)$$

where $V_R$ and $V_S$ are the nondimensional coefficients of variation (standard deviation divided by the mean) of strength and load respectively. Load ($\gamma$) and ($\phi$) factors are, therefore, dependent on the safety index and their respective coefficients of variation, i.e.

$$\phi = 1 - 0.7\beta V_R \qquad (9)$$

$$\gamma = 1 + 0.7\beta V_S \qquad (10)$$

Extensions of these basic equations have been reported for load combinations, nominal rather than mean values of variables and a lognormal (R/S) form of equation 3 rather than the normal (R - S) (1,16). Equations 1 - 10 give a theoretical basis for deriving safety factors based on probability theory and corresponding statistical parameters. These equations have already been used when introducing limit state formats which separate load and performance safety factors (2,4,13). Past experience must be incorporated through a code calibration process (18). This calibration cannot be overemphasized as the introduction of reliability formats must proceed with caution especially for "fleet" type systems such as highway bridges.

## Loading for Fatigue Design

The uncertainties in repetitive vehicle loading include truck dimensions, weight histograms and volume, possible changes during bridge lifetime of weight and volume and the likliehood of multiple presence of trucks on the bridge causing load superposition. For example, highway trucks vary in their number of axles and axle spacing, truck type, distribution of load to axles and gross weight. The frequency of different vehicle combinations depends on location (urban, suburban, rural and industrial, etc.), time, season and other economic factors.

Several studies of the different truck types (7,19) have concluded that reasonable accuracy in loading histories can be obtained if all the truck types are lumped into a few specific categories. This is because axle weights and total gross weights are the major factors affecting girder stresses while axle spacing and load distribution are of secondary importance. The fatigue behavior is also an averaging process depending on the load spectra rather than the specific occurrence of a relatively few load cycles. It has been suggested that even two truck types would be sufficient for a fatigue model lumping all vehicles as either tractor trailers or single unit trucks (19). As a justification, note that five axle tractor trailers often comprise on an interstate highway over half of the truck traffic stream.

Stress ranges at critical locations may be calculated as a function of bridge dimensions and the bending moments derived from the truck configurations. Uncertainty in stress analysis includes the distribution of total static bending moment to individual girders and also the dynamic response.

Fatigue properties of steel highway bridge components may be estimated from the considerable amount of experimental work done over the last few years (20). The tests show that fatigue life is dependent on stress range amplitude with different weld or attachment details behaving like stress concentrations. The results may be expressed as:

$$N S^3 = c \tag{11}$$

where: N - number of cycles to failure

S - stress range in constant amplitude sinusoidal loading

c - a constant depending on weld or attachment category

c may be expressed in terms of the stress range at 2 million cycles (any point on the fatigue curve could be used since a plot of log S vs. log N for the test data often exhibits no apparent fatigue life limit). This gives:

$$c = 2 \times 10^6 [S_{(2 \times 10^6)}]^3 \tag{12}$$

or substituting in equation 11 for the number of cycles to failure, N,

$$N = \frac{2 \times 10^6 [S_{2 \times 10^6)}]^3}{S^3} \tag{13}$$

Where: $S_{(2 \times 10^6)}$ - is the stress range amplitude for failure at two million cycles.

Laboratory tests also suggest that damage under random amplitude cycles can be estimated from the Miner linear cumulative damage theory. Fatigue failure occurs when the damage sum equal one, or:

$$\text{Damage, } D = V \sum \frac{f(S_i)}{N(S_i)} = 1 \tag{14}$$

Where: $f(S_i)$ - fraction of stress cycles at amplitude $S_i$.

$N(S_i)$ - number of constant stress $(S_i)$ cycles to failure (See equation 13).

V - total number of load cycles taken as the truck volume. This means each truck passage causes one cycle of load.

To further simplify calculations we assume that stress is proportional to vehicle weight (this is accurate for fixed truck dimensions). In order to normalize the data we designate a fixed dimension design truck, say of 72 kips, which causes a stress range, $S_r$ (see Figure 3). These assumptions imply a fixed relationship between stress range (S) and truck weight (W).

$$S = W \frac{S_r}{72} \tag{15}$$

Substituting in equation 14 the expression for N from equation 13, and replacing S from equation 15 gives:

$$D = \frac{V}{2 \times 10^6 [S_{(2 \times 10^6)}]^3} \sum f(W) \left(\frac{WS_r}{72}\right)^3 \tag{16}$$

Replacing the volume by the daily truck traffic (ADTT) and a 50 year life and rearranging terms gives for the damage, D:

$$D = \frac{50(365)\text{ADTT}}{2 \times 10^6} \left[\frac{S_r}{S_{(2 \times 10^6)}}\right]^3 \sum \left(\frac{W_i}{72}\right)^3 f(W_i)$$

$$= .0091 \left[\frac{S_r}{S_{(2 \times 10^6)}}\right]^3 L \tag{17}$$

Where: $L = \sum \left(\frac{W_i}{72}\right)^3 f(W_i)$

The summation, L, is denoted as the loadometer survey value and is a weighted average of truck loads using the cubic factor from the fatigue life data. Several state loadometer surveys were studied by Pavia giving loadometer values ranging from .28 to .52 with a value of .4 being slightly above average (21). This study showed the increase over time of both the load histogram and the truck volume. One disconcerting result was the inconsistencies between loadometer survey values in similar regions of the country or interstate highways passing through adjacent states. Also, there were differences in the high weight portion of the histograms between data taken from weigh station enforcement and data for load survey purposes (no penalties for violations). This is because overweight vehicles bypass enforcement stations. There is a need for more reliable data for bridge design purposes and this is discussed further below.

Solving equation 17 for the allowable stress range, $S_r$, for a damage, D = 1, gives

$$S_r = S_{(2 \times 10^6)} \left[\frac{110}{L(\text{ADTT})}\right]^{1/3} \tag{18}$$

The allowable stress range, $S_r$ in equation 18 is the stress range which for the weighted truck histogram given by L and volume ADTT just reaches the fatigue damage criteria at the end of 50 years. A safety check means that at each critical weld location the allowable stress range, $S_r$, should be compared to the design stress, $S_g$. The conventional approach is used here in which the gross bending moment is computed from the design vehicle increased by the impact factor. The girder moment is found by multiplying by the distribution factor. The design stress range, $S_g$, may therefore be written as:

$$S_g = \frac{M_R}{S_x} g\ h\ I \tag{19}$$

Where: $M_R$ - moment range calculated with a specified fixed wheelbase design truck weighing 72 kips and having average tractor-trailer dimensions as shown in Figure 3.

$S_x$ - girder section modulus

$g$ - stringer distribution factor depending on stringer spacing and deck configuration

$h$ - average headway or multiple presence factor

$I$ - impact factor

The factor (h) is used to incorporate the multiple presence of trucks on the bridge causing superposition. In a reliability format, the safety checks mean the comparison of design stress (equation 19) caused by the load with allowable stress (equation 18) based on the strength after determining suitable probability-based load and performance safety factors. These factors must be found by treating as random variables the strength terms, $S_{2 \times 10^6}$, L and ADTT and the load term, $M_R$, g, h, I and $S_x$. In this general reliability model the uncertainties, therefore, include load, highway bridge analysis and strength. Walker (22) has discussed the probability distribution of some of these parameters in computing the probability distribution of the bridge life. Knab, et al (23), have applied such distributions to the design of military bridges. For civilian highway bridges, the uncertainties in load and the limited data base require an approximate safety index code format to control the risk of premature fatigue during the expected bridge lifetime. Following the reliability theory described in equations 1 - 10, a margin of safety ratio, Z, is defined as

$$Z = \frac{\text{STRENGTH}}{\text{LOAD}} = \frac{S_r}{S_g} \tag{20}$$

The safety index (β) for this case has been defined from a lognormal format and leads to the following expression (1):

$$\beta = \frac{\text{Ln}\ \bar{S}_r - \text{Ln}\ \bar{S}_g}{\sqrt{V_{S_r}^2 + V_{S_g}^2}} \tag{21}$$

Where: $\bar{S}_r$ - mean allowable stress range

$\bar{S}_g$ - mean design stress range

$V_{S_r}$ - coefficient of variation of $S_r$

$V_{S_g}$ - coefficient of variation of $S_g$

The safety index (β) has a similar interpretation as in equation 6 except that it gives a precise probability value if the variables are lognormal rather than normal. [Since $S_r$ and $S_g$ are made up of the products of random variables a lognormal rather than a normal format is preferred.] Eliminating the square root term as in equation 7 and rearranging terms gives as a safety criteria:

$$\bar{S}_r e^{-0.7\beta V_{S_r}} \geq \bar{S}_g e^{+0.7\beta V_{S_g}} \tag{22}$$

Both $S_r$ and $S_g$ in equations 18 and 19 are related to the same 72 kip design vehicle. Equation 22 can be used as a safety check to derive respective load and performance factors.

Using equation 19, the mean girder design stress (load) is:

$$\bar{S}_g = \frac{\bar{M}_r \bar{g}\ \bar{h}\ \bar{I}}{\bar{S}_x} \tag{23}$$

and Load Coefficient of Variation

$$V_{S_g} = \sqrt{V_{M_f}^2 + V_g^2 + V_h^2 + V_I^2 + V_{S_x}^2} \tag{24}$$

From equation 18, the mean allowable Stress (strength) is:

$$\bar{S}_r = \bar{S}_{2 \times 10^6} \left[\frac{110}{\bar{L}\ (\overline{ADTT})}\right]^{1/3} \tag{25}$$

and strength coefficient of variation:

$$V_{S_r}^2 = \sqrt{V_{S_{2 \times 10^6}}^2 + \left(\frac{1}{3} V_V\right)^2 + \left(\frac{1}{3} V_L\right)^2} \tag{26}$$

Where $\bar{M}_R\left(V_{M_R}\right)$, $\bar{g}\left(V_g\right)$, $\bar{h}\left(V_h\right)$, $\bar{I}$, $\left(V_I\right)$ and $\bar{S}_x\left(V_{S_x}\right)$ are mean (coefficient of variation) of moment range, girder distribution, headway superposition, impact and section modulus respectively. $\bar{S}_{2 \times 10^6}$ $\left(V_{S_{2 \times 10^6}}\right)$, $\bar{L}\left(V_L\right)$, $\bar{V}\left(V_V\right)$ are mean (coefficient of variation) of constant amplitude fatigue stress at 2 million cycles, loadometer survey and volume respectively. Each of these statistical parameters must be estimated for the specified bridge traffic and section being checked.

From a practical code viewpoint, the means and coefficients of variation can be combined into discrete load and performance factors for various categories of roadway and weld attachments. Statistical data for carrying out the codification can be found from loadometer surveys, field measurements of stress histories and laboratory fatigue data. This information has been reported to code committees in a format which leads to more uniform safety levels (24).

In general, proposed changes in design specifications should not lead to drastic differences in design section values. This involves examining current structures which are believed to have good performance and computing their implied safety index. If these structures are generally considered not to be excessively over-designed, then some representative average of their safety indices becomes the target goal in a new, balanced risk code format (1,2).

For the fatigue design model the calibration of the reliability format uses equation 21 to calculate β for a number of different bridge configurations, traffic and weld combinations. This was done to determine the safety index, (β) implied in current design (21,25). A weighted value of the β's becomes the risk level in the code format with each load and resistance parameter calculated from equations 22 - 26. Typical β values using current design are found in the range of 1.5 to 3.0. An advantage in using equations 21 - 26 to derive code design factors is to achieve a more uniform safety level. This has been studied and the specific

results published but its presentation herein is beyond the present scope of this paper (21,25).

## Reliability Model for Strength Design

Many of the uncertainties in modelling fatigue loading are also found in describing strength behavior. For example, analytical and simulation studies have found the distribution of maximum bending moments using weight histograms, truck characteristics and multiple presence frequencies (10,19). These studies highlight the influence of these uncertainties on the peak distribution. Several important differences, however, should be considered between fatigue loading and a model for strength design.

1. The number of repeated loadings on a typical short or medium span bridge can easily exceed fifty million during its lifetime (i.e., ADTT > 2700 for a projected 50 year life). This causes formidable statistical problems to estimate the maximum loading distributions which are sensitive to the distribution tails used for multiple presence, truck weight, truck characteristics, impact, etc. The confidence in such distribution tails is small and casts doubt on the validity of these maximum load predictions.
2. A strength failure is often more severe than a fatigue crack so the safety index or risk level must be more conservative. This further limits the usefulness of simulation based on the current load data.
3. Predictions based on truck load "data" use measured or available information which may not correspond to maximum loads. The latter may, in fact, arise due to illegally overweight vehicles operating on the highway system.
4. The specified design load may also have political implications. If the specified design loads are the maximum expected loads, then these values may be cited to justify legislative increases in the legal load limits. For example, one load data study has shown, at least for axles, that the percent of illegal values remains almost constant and independent of the legal specified value (26). Marketing practices may encourage illegal loads so judgement and control must be exercised in raising design loads.
5. Despite the difficulty of predicting maximum loading the fact remains that few bridges exhibit failure due to a negative margin of safety (R - S) as illustrated in Figure 1. One explanation is the overload capacity contained in conservative analysis and design practices. Tests on recently constructed bridges show reserve strength several times in excess of the design load without encountering distress (8). For example, failure of steel beam-slab bridges is a progressive development under increasing load with large deformations, cracks and local buckling preceding collapse.

The uncertainty of load data coupled with the problems cited above make it difficult to derive appropriate safety factors for these solutions. Figure 4 shows three typical structural responses for bridge behavior. Damage vs. load is plotted in Figure 4a for a statically determinate or brittle structure which exhibits little reserve strength beyond a serviceability limit and failure is quite sudden. Figure 4b illustrates a response for a structure with moderate reserve strength which exhibits progressive damage as the load increases. Figure 4c shows a structure with large reserve strength capabilities and small amounts of damage until the load has exceeded several times the initial serviceability limit.

Figure 5 illustrates the load and response uncertainty superimposed on these damage vs. load curves. The expected damage cost (D) may be calculated by integrating over the load distribution ($f(\ell)$) as:

$$\text{Damage Cost,} \quad D = \int \left[ \int f_D\left(d \middle| L=\ell\right) dd \right] f_L(\ell) d\ell \quad (28)$$

where $f_D\left(d \middle| L=\ell\right)$ is the conditional damage frequency given a load $\ell$. The difficulties in finding precise distributions for the damage cost are apparent from the previous discussions of the load and strength uncertainties. This points, however, to designing for the whole range of responses from serviceability damages through collapse and even maintenance and rehabilitation. Some general conclusions are, however, obvious. Bridges, with behavior as in Figure 4a, show few signs of distress prior to high damage levels being encountered. These bridges are less likely to exhibit any warning signs under loads less than the vehicle loading causing significant damage. A bridge, as in Figure 4c would show signs of distress at loads significantly below the ultimate load. It would provide the safe options of strengthening the bridge or tightening the enforcement of load limits.

The following design approach is a general guideline. It is not intended that each bridge be studied this way. Rather, it suggests tools for code writers and researchers to classify groups of structures and provide more uniform reliability levels than currently exist.

1. Develop a safety index analysis ($\beta$) using a strength limit criteria. This would be used to derive element oriented load and performance factors as in the current AASHTO and other codes. Rather than categorizing this member design as the strength limit we should recognize that it is only another control on performance. It does not recognize the potential of the bridge to resist collapse or even large damage under major overloads. This would only be predicted by an ultimate load analysis to derive a damage curve as in Figure 4. It also implies that the definition of element strength is not critical and could either be elastic stress, plastic moment capacity, maximum strain, etc. Since the "damage" levels in this level of element strength or serviceability limit state are not severe, the safety index ($\beta$) need not be as high as for a true strength limit state.
2. Research on behalf of code writing agencies should perform ultimate nonlinear behavior analysis of various types of bridge structures. These analyses, supplemented by test data should lead to damage vs. load curves. The results should also show the distribution of reserve strengths beyond the serviceability limit state.
3. One possible additional input to codified design is a computed ratio of ultimate (U) to serviceability load (S). This should be incorporated in the strength performance factor as a system coefficient. This system performance coefficient should be in addition to the element strength factor derived from the conventional reliability theory described above. If the ratio (U/S) is close to one, as say, in a structure with important non-redundant elements, the system coefficient would be large, say at least 2.0. If the ratio of U/S is large, as in the slab-beam bridge where it may exceed 4 or 5, then the partial system factor may be much smaller, say 1.2.
4. These proposals contained herein are only preliminary conclusions, as further study is needed of full-scale bridge performance. In addition,

accurate load statistics must also be made available.

## Load Data

A major obstacle to a probability oriented specification for bridge strength is the relatively sparce load data available. The load history studies which accumulated data for fatigue design primarily measured stresses at critical attachment locations (6,7). Generally, these studies could not relate to the vehicle configurations, headways and axle loads causing the stresses being measured. While such detailed information is unnecessary for predicting repetitive fatigue loads, they are important for establishing design loads. The extensive strain-history study performed at Case Western Reserve University in Ohio measured some 20,000 trucks crossing on ten bridges and considered headway and truck type in the automated data acquisition and processing (27). Random gross vehicle weights, axle spacing and axle weights were not recorded.

The loadometer survey data has obvious inconsistencies since overweight vehicles avoid the survey stations because of penalties. This truncates the significant part of the load distribution. To improve the situation, a current project at Case has instrumented longitudinal girders of beam-slab bridges as load scales. The system obtains undetected weight information to produce an unbiased load data base.

During the last decade there have been several systems developed to weigh vehicles in motion (WIM). These WIM systems have utilized pavement scales or a section of roadway cut out and replaced with a slab held by instrumented supports. One difficulty with the WIM pavement scales is that the wheels are in contact with the scale for a very short time, say 5 - 10 milliseconds. The contact forces between tire and roadway undergo significant oscillations especially for typical roadway roughnesses encountered. Since the oscillation period is large compared to the time on the scale, the latter records a discrete value at some point on the oscillation which can be above or below the true static value. In a typical vehicle moving at high speeds, the tire force oscillations can easily exceed 50% or more of the static value. Hence, pavement scales require extremely smooth surfaces some distance in front of the scale and this surface must be periodically restored to maintain smoothness. If this is not done, then the best scale application would be low velocity operations, say at a busy survey station to segregate heavy vehicles for testing on the static scales. This does not solve the problem, however, of obtaining accurate data for vehicle load prediction.

The approach described here uses strain records of instrumented girders. A major requirement for accurate weight prediction is a description of the vehicle's dimensions. This is obtained by roadway sensors such as pneumatic tubes. A schematic of the operation is shown in Figure 6. The sensors provide the vehicle description while the analog strain data is digitized and stored in the computer for processing. To date, the processing is not carried out in the field but the data is stored on magnetic tape for subsequent computer processing.

Weight prediction is done as follows: The measured strain record ($M^*(t)$) is compared to a predicted signal ($M(t)$) based on the vehicle's configuration, axle spacing and bridge influence line ($I(t)$) (the latter can be obtained analytically or more accurately from calibrating with a vehicle of known axle spacing and weights). Thus the predicted signal is

$$M(t) = \sum_i A_i \, I_i(t), \quad i = 1, \text{ number of axles} \tag{29}$$

where $A_i$ are the unknown axle weights as shown in Figure 6. An error difference function, E, may be calculated as

$$E = \int_{\text{time}} [M^*(t) - M(t)]^2 dt \tag{30}$$

where the integration is over the time taken by the truck to cross the bridge. Substituting equation 30 for the predicted signal $M(t)$ gives:

$$E = \int [M^*(t) - \sum A_i I_i(t)]^2 dt \tag{31}$$

Minimizing the error (i.e., least square fit) gives a linear set of equations to solve for $A_i$.

This procedure has been tried in the Case Weigh in Motion system with promising results for weighing trucks in an undetected manner. The actual cost per bridge-scale is quite small and can utilize reuseable strain transducers. The investment is primarily in the electronic recording and processing equipment which can be transported to several sites. Further effort is underway to improve the accuracy of the system and to expand its usefulness.

Some examples of the results are shown in Tables 1 and 2. These show the weight obtained at a survey station and the values predicted by the Case WIM system. Table 2 shows predicted weight histograms compared to data from a load survey station and a state loadometer report. These were not taken at the same time so only relative comparisons can be made. The loadometer report which did not involve enforcement showed 11% of the five axle vehicles in excess of the legal limit while the Case WIM showed 15%. The load survey station which involved enforcement reported no overweight vehicles for the period observed. A field system to perform such weigh in motion measurements has been assembled at Case and delivered to FHWA in March, 1978.

## Conclusions

Structural reliability theory has been used as an analysis and design aid for incorporating probability and statistics in characterizing uncertainties in bridge load, behavior and capacity. Safety and performance factors can be derived to provide a uniform level of risk against unserviceability. Coupled with past experience and judgement, reliability methods can be used by code writers to introduce more rational and optimal code formats such as limit state design.

A detailed presentation of the fatigue design load indicates that uncertainties in vehicle weights, volume, multiple presence, impact, girder distribution, fatigue life and section properties can be incorporated in safety factors to achieve more uniform risk against premature fatigue cracks. An extension of the load prediction model to strength design presents difficulties because ultimate strength behavior is quite complex. A proposal is made to consider safety checks for strength on a serviceability basis with a system safety coefficient established to represent ultimate behavior. System coefficients would be prepared by code researchers for various types of bridges to represent the reserve capacity existing before significant damage occurence.

Uncertainty in load data is a major limitation in introducing reliability-based design. A project is described for undetected weighing of vehicles in motion by using instrumented highway bridge girders as load scales to predict axle and gross weights. Preliminary data reported shows the feasibility of this approach.

Acknowledgements

The author wishes to acknowledge contributions of his colleagues in carrying out various phases of this work and the support of sponsoring organizations. The development of the fatigue model and study was helped by R. Garson and A. Pavia in projects supported by Ohio Department of Transportation (ODOT) and the Federal Highway Administration (FHWA). Data for the fatigue study was obtained in a field measurement project supported by ODOT and FHWA and assisted by Professor George Goble. The Case Weigh-in-Motion study was supported by FHWA.

The contents of this report reflect the views of the author, who is responsible for the facts and accuracy of the data presented. The contents do not necessarily reflect the official views or policy of the State or the Federal Highway Administration. This report does not constitute a standard, specification or regulation.

Figure 1. Fundamental reliability model - weaving of load and strength distributions to obtain risk ($P_f$).

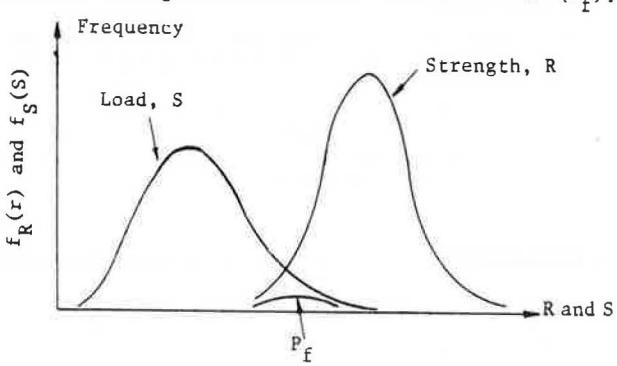


Figure 2. Illustration of total cost, initial cost and failure cost versus risk.

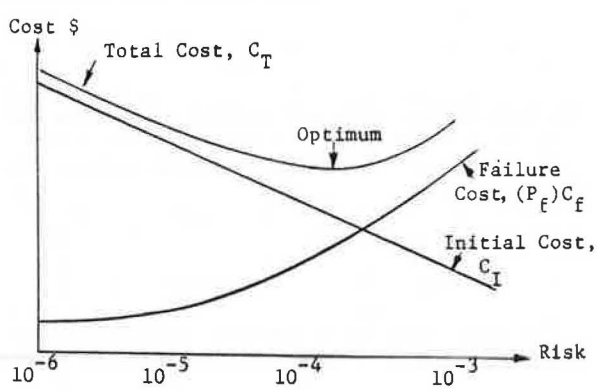


Figure 3. 72 kip tractor trailer with average dimensions and load distribution.

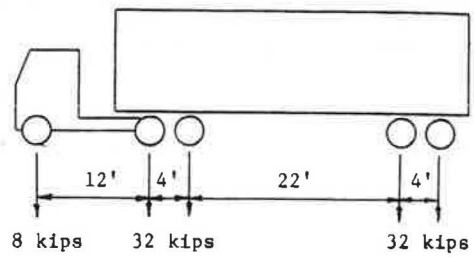


Figure 4. Illustrations of bridge damage versus load response curves.
(a) "Statically determinate or brittle" behavior.
(b) Moderate reserve strength.
(c) Large reserve strength.

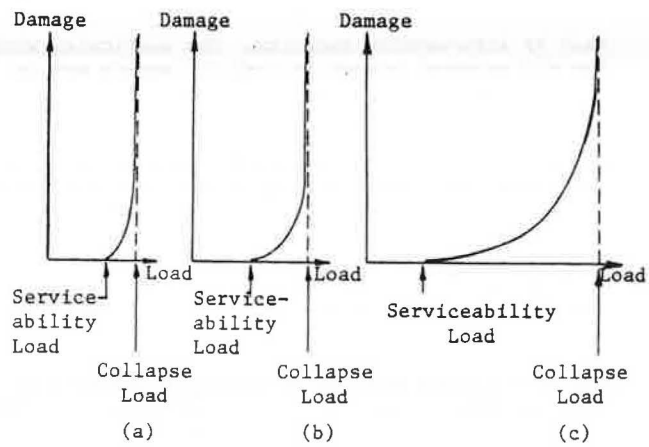


Figure 5. Illustration of bridge damage versus load with damage and load frequency distributions.

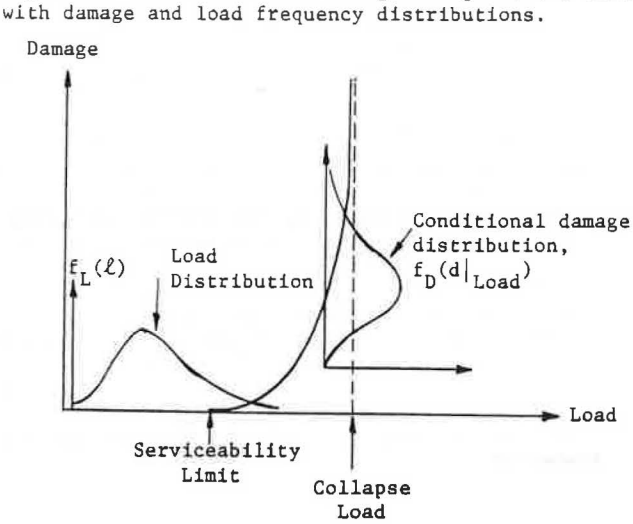

Figure 6. Schematic layout of weigh in motion system using instrumented bridge girders.

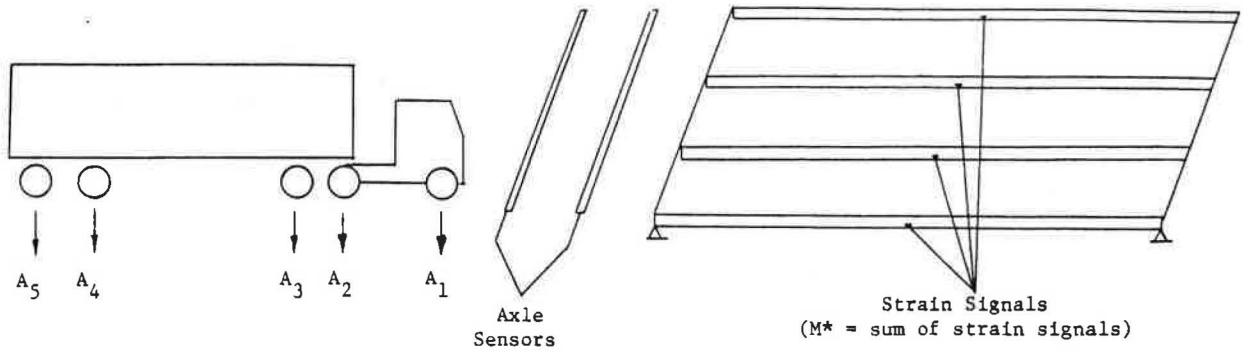


Table 1. Examples of WIM predictions and survey station weights.

| Case | Description | WIM Prediction Gross Weight (kips) | WIM Prediction Rear Tandem (kips) | Survey Station Gross Weight (kips) | Survey Station Rear Tandem (kips) |
|---|---|---|---|---|---|
| 1 | 4-axle delivery truck | 40.2 | 16.5 | 40.3 | 15.6 |
| 2 | 5-axle tractor trailer | 75.5 | 33.5 | 67.7 | 30.4 |
| 3 | 3-axle car carrier | 49.1 | 17.2 | 50.3 | 18.2 |
| 4 | 5-axle open trailer | 26.7 | 7.8 | 29.5 | 8.9 |

Table 2. Comparison of WIM predictions and survey station gross weights for tractor trailers.

| Gross Weight (kips) | Cumulative Percent State Loadometer Survey* (Percent) | Nearby Survey Station** (Percent) | WIM Prediction*** (Percent) |
|---|---|---|---|
| < 20 | - | - | 1 |
| 30 | 8 | 13 | 20 |
| 40 | 35 | 31 | 47 |
| 50 | 43 | 48 | 53 |
| 60 | 54 | 64 | 62 |
| 70 | 70 | 80 | 76 |
| 80 | 89 | 100 | 86 |
| Legal Limit | | | |
| 90 | 97 | | 93 |
| 100 | 100 | | 98 |
| > 100 | 100 | | 100 |

*Data from variety of state roads.
**Data taken in proximity to but not precisely correlated to WIM data.
***132 vehicle samples.

## References

1. A. Ang and C.A. Cornell, Eds. Modern Concepts of Structural Safety and Design. Journal of the Structural Division, ASCE, Sept. 1974.
2. T.V. Galambos. Proposed Criteria for Load and Resistance Factor Design of Steel Buildings. Report 45, Civil Engineering Department, Washington University, St. Louis, May, 1976.
3. Probabilistic Design of Reinforced Concrete Buildings. Publ. SP-31, ACI, Detroit, 1972.
4. S. Fjeld. Reliability of Offshore Structures. Paper OTC 3027, Offshore Technology Conference. Houston, Texas, 1977.

5. Standard Specifications for Highway Bridges. AASHTO, American Association State Higway Officials, 1969.
6. R.F. Varney and C.F. Galambos. Field Dynamic Loading Studies of Highway Bridges in the United States, 1948-1965. Highway Research Record 76, Highway Research Board, 1965.
7. C. Heins and C.F. Galambos. Highway Bridge Fatigue Tests in the United States, 1948-1970. Public Roads, Feb. 1972.
8. D.W. Goodpasture and E.G. Burdette. Comparison of Bridge Stress History Results with Design-Related Analyses. Highway Research Board 428, Washington, D.C., 1973.
9. H. Bissell, D. Cramer and R. Eckert. Forecasting of Heavy Loading Patterns on Highway Bridges. Contract FH-11-7314, Kelly Scientific Corp., Washington, D.C., June 1970.
10. D. Harman and A.G. Davenport. The Formulation of Vehicular Loading for the Design of Highway Bridges in Ontario. U. of Western Ontario, London, Dec., 1976.
11. H.K. Stephenson. Highway Bridge Live Loads Based on Laws of Chance. Proceedings, ASCE, Vol. 83, July 1957.
12. Colloquium on Highway Bridge Loading. International Association of Bridge and Structural Engineers, Cambridge, England, April 1975.
13. R.A. Dorton and P. Csagoly. The Development of the Ontario Bridge Code. Ontario Ministry of Transportation and Communication, Downsview, Oct. 1977.
14. C.W. Peterson. Bridge Rating by Limit State. Report prepared for CSA S6, Province of Alberta, Bridge Branch, Dec. 1976.
15. C.A. Cornell. A First Order Reliability Theory of Structural Design. Structural Reliability and Codified Design, Ed. N. Lind, SM3, U. of Waterloo Press, 1970.
16. L. Esteva and E. Rosenbleuth. Use of Reliability Theory in Building Codes. First Intl. Conf. on App. of Stat. and Prob. to Soil and Struct. Engineering., U. of Hong Kong, Sept. 1971.
17. M.K. Ravindra, N.C. Lind and W.C. Siu. Illustration of Reliability Based Design. Journal of the Structural Division, ASCE, Sept. 1974.
18. W.C. Siu, S.R. Parimi and N.C. Lind. Practical Approach to Code Calibration. Journal of the Structural Division, ASCE, July 1975.
19. F. Moses and R.C. Garson. New Procedure for Fatigue Design of Highway Bridges. TRB Research Record 507, 1974.
20. J.W. Fisher, K.H. Frank, M.A. Hirt and B.M. McNamee. Effect of Weldment on the Fatigue Strength of Steel Beams. NCHRP Report 102, HRB, 1970.
21. F. Moses and A. Pavia. Probability Theory for Highway Bridge Fatigue Stresses-Phase II. Final Report, Case Western Reserve University, Ohio, Department of Transportation, Aug. 1976.
22. W.H. Walker, Loading Histories. ASCE Specialty Conference on Metal Bridges, St. Louis, Nov. 1974.
23. L.I. Knab, A.H.-S. Ang and W.H. Munse. Reliability Based Fatigue Design Code for Military Bridges. ASCE Specialty Conference on Metal Bridges, St. Louis, Nov. 1974.
24. F. Moses. Comprehensive Fatigue Design Procedures. Presentation for AASHTO Bridge Committee Meeting, 1975.
25. F. Moses. Bridge Loading Spectra and Fatigue Design. Presented at ASCE National Meeting, New Orleans, April 1975.
26. R. Winfrey and others. Manual of Procedures for Conducting Studies of the Desirable Limit of Dimensions and Weights of Motor Vehicles. Report No. FHWA-RD-73-68, July 1970.
27. F. Moses, G.G. Goble and A. Pavia. Applications of a Bridge Measurement System. Transportation Research Record 579, 1976.