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Probabilistic Approaches to Bridge Design Loads

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Load uncertainties form a major part of the probabilistic basis for structural design. This paper is part of a state-of-the-art report on probabilistic design for bridges. The emphasis is on vehicle-induced loads for short- and medium-span bridges. Unlike environmental loads, vehicle loads can evolve according to imposed regulatory controls. Loads for railway bridges are relatively well controlled, and methods for deriving probabilistic load factors for repetitive spectra (fatigue) and maximum lifetime loading (strength) have been presented. Statistical data for highway bridges, however, are relatively sparse, especially for extreme vehicle weights and spacing configurations. Computer simulation and convolution models for deriving probabilistic load factors for strength are described, and safety indices for fatigue—including uncertainties in vehicle weights, truck dimensions, impact, headway, volume, girder distribution, and material fatigue life—are also presented. These probabilistic approaches are limited by the available data base and the continuing growth in truck loads. The paper also presents a system-reliability approach to model behavior beyond the simple element capacity check. Bridge damage versus load curves should be investigated for different geometries and configurations. Superimposing load and resistance probability distribution will provide damage costs for deriving optimum load factors. Because load growth would be partially absorbed in the bridge performance range between initial element failure and extreme damage, design ductility and redundancy need lower factors. Methods for establishing these goals are discussed.

An important part of a probabilistic approach to bridge safety is describing loads, which are caused by construction, erection, fabrication, settlement, temperature, traffic, and environmental forces. The AASHTO specification (1), for example, lists 10 loading combinations. Significant research under the general heading of structural reliability has described various load phenomena and different load and combination factors (2). For example, environmental forces such as wind, waves, earthquakes, and flooding are often described in terms of statistical parameters such as return periods (3). The major emphasis here will therefore be on a probabilistic framework and description of traffic loadings on bridges, especially short- and medium-span highway and railway bridges where the most uncertain load phenomenon is the live-load effect of traffic.

This paper describes the uncertainties of traffic loading and analyzes load spectra for fatigue and maximum lifetime load for a strength design. A careful distinction is made between load pattern and load factor. Both highway and railroad engineers have devised theoretical load patterns to provide envelopes for bending moment and shear that arise from existing or expected vehicle combinations. For example, AASHTO specifies a design vehicle with a variable wheel base intended to envelop moment and shear for simple and continuous spans over a wide range of lengths (1). Similarly, the theoretical Cooper loading is used for railway bridges to simplify the "bookkeeping" associated with checking strength and serviceability at various locations in a bridge. Other load patterns that provide better consistency for bending and shear response have also been invented for longer spans (4).

Load factors, however, as distinguished from the load pattern, are a quantity by which the calculated load effects are multiplied to provide safety margins that cover uncertainties such as high loads from heavy, closely spaced vehicles; approximations in the statistical analysis; and dynamic response. Further, as part of a reliability analysis, load factors can incorporate uncertainties in judgment, analysis, and failure consequences. A basic feature of an analysis of reliability or probabilistic safety is that load factors relate directly to load uncertainties that have different factors appropriate for dead load, live load, wind, and other forces. Reduction in loads is needed when a multiplicity of load effects combine. At the present stage of reliability applications, a measure of risk such as a safety index β is usually employed (5). For a simple check of strength capacity, the safety index can be written in the popular additive form as (6)

$$\beta = \text{mean safety margin/variance of safety margin} = (\bar{R} - \bar{S})/(\sigma_R^2 + \sigma_S^2)^{1/2} \qquad (1)$$

where $\bar{S}(\bar{R})$ and $\sigma_s(\sigma_R)$ are the mean and standard deviation of the load (strength) respectively. This format (2) gives rise to a load factor γ as

$$\gamma = 1 + \alpha\beta V_S \qquad (2)$$

where α is the separation factor arising from the square-root quantity in Equation 1 and V_s is the nondimensional load coefficient of variation (standard deviation divided by the mean). In code writing, each partial factor (load and resistance, or performance factor) is a function of its respective coefficient of variation V and a common safety index β that produces decreasing risk with increasing value. Load-combination factors may also be formulated (2) by extension

$$S = S_1 + S_2 + \ldots + S_i + \ldots + S_n \qquad (3)$$

where S_i is the ith load effect. Each load factor γ_i can be expressed as

$$\gamma_i = 1 + \alpha_i \beta V_{Si} \qquad (4)$$

The separation factors α_i are functions of the number of load terms and their variances. In practice, calibration reduces the number of different load-combination factors. This results in a tabular form similar to the AASHTO load table.

In order to apply these probabilistic methods, it is necessary to obtain mean loads and their variances. This will be discussed separately in relation to railway and highway loads for both fatigue (repetitive) spectra and strength (maximum loading). In addition, because of the nature of vehicle loadings, which are controlled by users and subject to future changes after construction, strategies are needed to specify design loads and ultimately to optimize the material resources in bridge construction.

RAILWAY LOADS

In the context of probabilistic load descriptions, railway bridges are unique among structures for several reasons. The design, construction, inspection, and operation are controlled by the owner or railroad company, which can specify permissible loads to limit force effects and control speed to limit dynamic impact. Thus, the nature of load uncertainties differs from most structures and even from highway bridges that have uncontrolled access. The relatively high live-to-dead load ratio also means that, in design and rating of short- and medium-span bridges, the most significant loads are the live-load (train) effects.

Uncertainties in Load Effects

Train Configuration

To simplify force calculations, the theoretical Cooper E loading system was introduced before 1900. It represents a double-headed steam locomotive followed by a uniform load. Although in present configurations the maximum stress is produced by a string of heavy cars rather than a locomotive, the Cooper's loading has survived as a standard. Drew, for example, has reported (8) that some of the tank cars, flat cars, hopper cars, and other special cars that carry unusual loads as large as 34 000 kg (75 000 lb), spaced at 1.4-m (4.5-ft) centers that produce Cooper equivalents as large as E93 for 12- to 14-m (40- to 45-ft) spans and only E77 for 2.5-m (8-ft) spans. It is especially in repetitive fatigue spectra that these Cooper loadings are unrealistic (9).

Dead Load

In contrast to other structural applications, the dead-load uncertainty can be significant especially for ballast-floor spans. Byers (10) suggests a 15 percent coefficient of variation for errors in estimating unit weight and ballast depth during the bridge life; compare this to a 3 percent variation suggested for the uncertainty in estimating dead weights for bridges in service.

Calculation of Load Effect

Because of the geometry, main floor beams and stringers have additional strength that is usually ignored in design calculations. Based on field measurements, stresses are 80-90 percent of calculated values, depending on span and type of deck layout. For longer spans, Byers (10, 11) suggests a mean ratio of 0.93 of actual to calculated forces and a coefficient of variation of 15 percent.

Impact

The load-effect magnification resulting from vehicle velocity compared to a static or crawl speed has been designated as impact. An analysis of field tests by Byers indicated that impact factors were normally distributed and showed mean and standard deviation increasing with speed. Individual variations are large because impact is a function of roll, speed, and track effects that produce results with variable phases. Byers, in analyzing a number of high-speed tests at 113-145 km/h (70-90 mph) and spans of 18-27 m (60-90 ft), found mean increases of 14 percent and coefficients of variation of 0.61.

Strength Design (Maximum Lifetime Loading)

The live-load effects (S) for a limit-state format are

$$S = L(1 + I) + D \qquad (5)$$

where

L = load effect induced by vehicles,
I = impact, and
D = estimated dead load.

For a safety index, we need the mean load $\bar{S}$ and standard deviation σ_S (Equation 1). Using the values cited above as illustration gives as a first-order approximation

$$\bar{S} = \bar{L} + \overline{LI} + \bar{D} = 0.93L_c + (0.93L_c)(0.14) + D_c \qquad (6)$$

where L_c and D_c are the calculated live- and dead-load effects. The ratio of actual load to calculated load is used only for the live load. Similarly, for the variance (σ_S^2) the result is

$$\begin{aligned}\sigma_S^2 &= \sigma_L^2 + \bar{L}^2\sigma_I^2 + \sigma_L^2\,\bar{I}^2 + \sigma_D^2\\ &= [0.15(0.93L_c)]^2 + [0.93L_c(0.61)0.14]^2\\ &\quad + [0.15(0.93L_c)0.14]^2 + [0.15D_c]^2 \qquad (7)\end{aligned}$$

The load factor (γ) is found by substituting in Equation 4. Equation 7, however, ignores statistical correlation between live load and analysis uncertainties.

Fatigue Design

Welded bridges have had considerable application in recent years, but their designs were often limited by fatigue considerations. A general approach to formulating a bridge fatigue specification is as follows (12):

1. Compute the stress range caused by a design-load pattern;
2. Transform the range into a fatigue-damage contribution with the constant amplitude S-N curve for the appropriate fatigue detail;
3. In a similar manner to step 1, transform the load spectrum (histogram of load occurrences) into a damage spectrum; and
4. Estimate the fatigue life using Miner's rule and the annual number of load occurrences, or establish the design stress range to reach a desired life.

This approach, accepted by both the American Railway Engineering Association (AREA) and the American Association of State Highway and Transportation Officials (AASHTO), can be calibrated to give consistent expected lives for different details. This is further elaborated below for highway bridges and is also given in a safety-index format. The major existing problem for application to railway bridges is that the theoretical Cooper load pattern is unrealistic for the most severe fatigue-inducing load cycles now operating (9). More data are needed on actual train make-ups in relation to fatigue, although some studies are reported to be under way (13).

In contrast to highway bridges, few cases of railway bridge stress monitoring have been done recently under traffic conditions. Medium- and long-span bridges are not usually fatigue sensitive, because each train (rather than a single car) causes one stress cycle—provided the load is relatively uniform. One irony is that, in efforts to move large loads, heavy cars are often separated by empty cars to reduce maximum static effect; this, however, increases the number of fatigue stress cycles.

Rating

An important process in bridge safety is the inspection and rating of bridges for determining safe load-carrying capacity in their existing conditions. Better knowledge regarding the strength and static and dynamic behavior of an existing structure is needed. Furthermore, rating covers a much shorter period than the typical design life. For these reasons, the safety index will be higher in a rating analysis. This does not mean that the strength capacity is higher—it may be reduced by material loss—but only that there are fewer overall uncertainties about load and resistance. Byers (10) reported for one example a 23 percent increase in the safety index for rating compared to design.

HIGHWAY LOADS

Highway vehicles vary widely in their gross weight, axle spacing, and distribution of gross load to axles. The uncertainties in vehicle-load effects include location in lane, velocity (impact), and likelihood of many trucks on a bridge at once, which could cause load superposition.

The frequency of different vehicle combinations depends on locale (urban, industrial, etc.), time of day, season, and various economic factors. Because codes evaluate component capacities rather than total performance, the load effect in typical beam elements is also affected by uncertainties in the simplified stringer and longitudinal beam distribution factors. The latter are usually based on simultaneous lane occupancy, which may occur for maximum static loads but is conservative with respect to fatigue spectra.

Truck loading is a random phenomenon for which probabilistic models and statistical data are useful. Unlike railway loads, maximum highway loads are not determined by imposed limits but are based on random combinations. Load studies generally have been of two types: (a) analytical or computer models that predict random occurrences of vehicle combinations including weights and spacings and (b) measured bridge stress history studies that obtain stress spectra. The first type is used for predicting maximum lifetime loading and fatigue spectra, while the field results have been used for predicting fatigue life. Although analytic load models should be correlated with field stress measurements, limitations in equipment have made it difficult to correlate random vehicle-loading combinations with the measured bridge responses. Some recent work has produced bridge-oriented weigh-in-motion instrumentation that could contribute more data (14).

Load-Prediction Models

Probabilistic models for highway loading are complex because the maximum lifetime loading is the largest load of many millions of unique load applications. Hundreds of thousands of significant stress cycles are needed for fatigue damage, and such load spectra are affected by traffic patterns, future growth in vehicle weights and volumes, and vehicle and bridge geometry. Thus, prediction models contain a large number of random variables in order to produce meaningful results of high statistical confidence.

Published studies of traffic loads have ranged from application of stochastic processes to semiempirical methods. Tung (15) attempted a purely analytical approach by modeling truck-traffic effects as a filtered Poisson process and calculated peak probability densities and crossing rates. Ditlevsen modeled truck spacing with exponential distributions and calculated probability functions for bridge response (16). These analytical approaches require simplifications in truck dimension, axle weight, and headway spacing that severely limit their use for short and medium spans where live load is most significant. A more practical approach is numerical convolution or Monte Carlo simulation with a limited number of random variables but realistic distributions for those variables believed to be important.

In the convolution approach, the response distribution is obtained by summing over the multiple distributions of the independent variables. Unfortunately, such computations increase exponentially as the number of variables increases. Monte Carlo simulation, on the other hand, generates sequences of random loads that obey the statistical laws of the governing variables. It is especially useful for obtaining means and variances, but many trials are required to obtain small probability values with any high degree of confidence.

In an extensive application by Fothergill, Lee, and Fothergill (17), simulated truck-traffic patterns were developed as a chain of traffic events until arrival on the bridge and response were calculated with a dynamic finite-element structural analysis. This detailed calculation severely limits its ability to produce a load-effect spectrum.

Moses and Garson (18, 19) used the convolution approach after identifying the major ingredients in the load pattern. This included truck type (which could be adequately represented by two categories, tractor-trailer and single units), gross weight distribution, and headway. Although the program could model as many as three trucks simultaneously, parameter studies for typical spans showed that only two trucks acting simultaneously affected fatigue spectra. Vehicle dimensions, axle fractions, and dynamic amplification were held deterministic in the model. The load spectra compared with measured bridge stress histories from several different states agreed well (18). Headway was modeled as an exponential distribution, but measured headway data obtained from field tests were used later on (20).

Further work along these lines by Christiansson in Sweden included nondeterministic lateral location of the vehicle and dynamic amplification (21). Harman and Davenport (22) reported prediction models that used Ontario data to develop the recommended load factors in the proposed Ontario bridge code (23). An actual truck sample was simulated along with an assumed headway-spacing model. Their simulation predicted the distribution of the largest bridge response in 50 years for representative spans and configurations.

Live-load uncertainty has usually been modeled for short spans. For long spans [longer than 125 m (400 ft)] dead load dominates except in some special cases such as single box girders and suspension and cable-stayed bridges that can be sensitive to errors in live loading.

Studies by Navin, and others (24) have developed a simulation program for long spans that considers the severity of traffic stoppages, number of lanes, and percentage of cars and trucks. Their load predictions were reported to be less severe than those suggested by Ivy and others (25) or Asplund (26) in the form of "average load per unit length" versus loaded length for a specified return period (27).

Data Requirements

The need for further truck-weight data and field measurements was reported in all the studies cited. Insufficient data exist for extreme truck weights because such vehicles often operate in violation of legal limits. Regarding the headway spacings, the Poisson arrival model and exponential distribution have been verified for automobile traffic and are therefore usually used.

Tests on 10 separate bridges indicated that the model was conservative because trucks have a natural tendency to maintain longer distances apart and avoid wide speed variations that lead to overtaking (20). Data are not yet available, however, to correlate speed or headway with weight.

APPLICATION OF HIGHWAY LOAD MODELS

Fatigue

Recent implementation of fatigue specifications for highway bridges has combined load-spectra models, laboratory data on welded attachments, and field observation of stress histories and bridge behavior (12). In addition to the assumptions cited above for railway bridge fatigue, the following assumptions are made for highway bridge fatigue.

1. Each truck crossing produces one cycle of stress range amplitude proportional to vehicle weight. Design-vehicle characteristics of axle spacing and axle fraction are usually used.
2. The number of load cycles equals the expected truck volume.
3. Loadometer studies provide the gross weight distributions that are transformed into load-effect spectra.
4. The relationship between calculated bending stresses and actual experienced stresses can be estimated from field stress studies. This reflects vehicle occupancy of one lane rather than all lanes and the conservativeness of the stringer distribution factor. Typical values for longitudinal beams are 0.7 for a single lane or 0.35 for two lanes (20).

These assumptions lead to an expression for the fatigue damage D as

$$D = V/A\Sigma S_i^3 f(S_i) \tag{8}$$

where

- V = lifetime truck volume or average daily truck traffic (ADTT) × 365 × number of years of service,
- $f(S_i)$ = frequency of stress range (S_i), and
- A = function of the fatigue-life intercept of the weld attachment.

The cubic exponential factor appearing in Equation 8 arises from the stress range fatigue life (S-N) plots that have a slope of 3 on a log-log graph. In this generally used code, safety is maintained by choosing stress-range intercepts with a 95 percent confidence level for 95 percent survival (12). To place the fatigue code in a reliability framework, Moses and Pavia (29, 30) studied means and coefficients of variation of the random variables controlling the fatigue-life estimate. These variables included the moment range (which depends on vehicle dimensions), headway, stringer distribution, impact, volume, loadometer weight survey, and the weld-attachment stress-range amplitude. These calculations can be summarized as follows. The design girder stress range (S_g) was expressed as

$$S_g = (M_R\, g\, h\, I)/S_x \tag{9}$$

where

- M_R = bending-moment range due to a load pattern defined as a five-axle vehicle with average axle dimensions and axle weight distribution (18),
- g = stringer distribution,
- h = headway allowance for closely spaced trucks,
- I = impact, and
- S_x = section modulus.

The corresponding strength or resistance was expressed from Equation 8 by setting the damage D equal to one. To simplify calculations, the stress range S_r is evaluated as corresponding to a gross vehicle weight of 32 600 kg (71 700 lb). Therefore, assuming that stress range S_i is proportional to gross vehicle weight GVW_i gives

$$S_R/72 = S_i/GVW_i \tag{10}$$

The truck-weight survey in terms of equivalent damage L is defined as

$$L = \sum_i [(GVW_i)/72]^3\, f(GVW_i) \tag{11}$$

where $f(GVW_i)$ is the frequency of trucks with GVW_i. Substituting Equations 10 and 11 into Equation 8 and using D = 1 gives the corresponding stress range S_r, which produces damage for weight distribution L, cycles V, and fatigue attachment A.

$$S_r = [A/LV]^{1/3} \tag{12}$$

The safety-index analysis is conveniently obtained by defining a margin of safety ratio Z [log-normal format (2, 31)] as

$$Z = S_r/S_g \tag{13}$$

so that the safety index β (2) becomes

$$\beta = (\mathrm{Ln}\bar{S}_r - \mathrm{Ln}\bar{S}_g)/(V_{S_r}^2 + V_{S_g}^2)^{1/2} \tag{14}$$

Evaluation of β requires means and coefficients of variation of moment range (M_r), girder distribution (g), headway effect (h), impact (I), loadometer survey (L), volume (V), and material factor (A). Representative values were obtained from field measurement studies (20), traffic reports (29), and laboratory tests (32). For example, volume and loadometer surveys were assigned coefficients of variation of 6 and 10 percent, respectively. Moment range (vehicle dimension uncertainties), girder distribution (mean = s/14.7 for a wheel load), impact (mean = 11 percent), and headway (mean = 6 percent) were assigned coefficients of variation of 11 percent, 13 percent, 11 percent, and 6 percent, respectively (29, 30). A study of fatigue-life laboratory test data showed a 10 percent coefficient of variation.

Making these substitutions for a variety of bridge configurations and traffic and weld categories showed that current designs have typical safety indices (β) in the range of 1.5 to 3.0, which is consistent with other structural experience (29, 30). Such studies are in the form of a calibration that tries to achieve uniform reliability levels without causing significant changes from current design. One conclusion, for example, is to make allowable stress ranges continuous functions of truck volume rather than discrete categories as is now done (28). Another suggestion is to use representative vehicle dimensions in the fatigue calculations. These methods of deriving fatigue-safety indices can be extended to strength design as discussed below.

Strength Design

Strength design needs predictions of maximum lifetime loading, so the sensitivity to distribution tails is greater than for fatigue spectra. Available load data are also less adequate for prediction because the maximum load may be from illegally operating vehicles or future legal load limits and changes in vehicle design.

One aspect of load specification is the load pattern for calculating force effects. It should be simple and provide uniform safety margins. The second aspect is the load factor for accounting for overloads, growth in load over time, uncertainties in analysis, and consequences of failure, that is, the usual safety requirements for a load factor in a code.

Load Pattern

Various load patterns are used in different codes around the world (4). The important point is not how large the load is but how consistent it is, compared to field observation under extreme load, in producing design bending moment and shear envelopes. For example, the probability of all traffic lanes being simultaneously loaded is small, especially when impact is considered. This calls for a lane-reduction factor. Further, the probability is also small of many extremely heavy vehicles moving in a closely spaced train. This means that load effect should decrease with loaded length.

Some commonly used load patterns are the design vehicle and a uniformly distributed load (UDL). The AASHTO code uses a design vehicle with a variable axle spacing. This increases shear and moment on shorter spans and increases negative moments at internal supports on continuous spans. For longer spans, AASHTO bridges are controlled by a UDL plus a concentrated load developed to represent a train of trucks. No span-length reduction factor is specifically used, although a single concentrated load alone partly accomplishes this purpose. Another load pattern is the proposed Ontario bridge code, which uses an 18-m (60-ft) five-axle vehicle plus a UDL for long spans (23).

In the United Kingdom and several other countries, a design loading consists of a concentrated load plus a UDL whose intensity is a function of the loaded span length. A report by Thomas (4) surveyed bridge loadings in some 18 countries and showed the AASHTO design forces to be the lowest of any country's. This accounted, however, strictly for the load pattern but not the overall safety margin, which must also be based on the stringer distribution factors and the material-safety margin. The relatively light 32 600-kg (71 700-lb) AASHTO design vehicle may be intended more to reduce pressures than to increase legal load limits. More research is needed to make load patterns representative of current maximum loading conditions.

Load Factor

In the load-factor design concept for bridges, the factored maximum strength (ϕ) of each bridge element must exceed factored forces (γ) on this element caused by the design load (1).

$$\phi \geq \gamma[\beta_D D + \beta_L(1 + I)] \qquad (15)$$

For the AASHTO provisions, γ values were recommended after a calibration process considered different bridge configurations and span lengths (33). The 1.3 value provided lighter structures than working-stress design for longer span lengths. This calibration with past design practice was done without actually involving load statistics or probabilities of maximum response. LFD reapportioned material within the total system of bridges to realize consistent safety margins and, indirectly, more uniform levels of reliability.

The next step in the reliability hierarchy is to make the load factor directly dependent on load statistics and a safety index β. If we feel confident in the live-load data and the ability to control and enforce load limits, we could formulate a probabilistic approach similar to that for railway bridges. The live- and dead-load effects include

$$S = L(1 + I) + D \qquad (16)$$

where mean $\bar{S}$ and variance σ_S^2 are defined as in Equations 6 and 7. The load-simulation models described above should be used to give mean $\bar{L}$ and variance σ_L^2. Such an analysis has been performed for the Ontario code cited above (22, 23). Impact values should be based on field measurement tests, although there is as yet little correlation of impact with extreme live-load conditions. Reasonable values might be $\bar{I}$ equals 20 percent and V_I equals about 30 percent. The coefficient of variation of dead load should be typically around 5 percent for steel and precast concrete and somewhat higher for poured-in-place concrete and deck materials.

Representative safety indices can be derived by the calibration process illustrated above in the fatigue calculation or in the load-resistance factor design (LRFD) format for steel buildings by Galambos (34). A code could also have indices that depend on failure consequences.

There are several important limitations in such an approach to deriving probability-based load factors. Load magnitudes are evolving over time as legal limits and economics of heavy-vehicle design and operation change. This means the last 50 years may only be a guide for future designs, not a direct source of data as, say, in structures subject to environmental load. Code calibration may also mask the fact that bridges and codes deliberately contain conservative factors to account for load increases and that the safety indices will not be comparable to building structures. In addition, there is still a limited data base on extreme vehicle loads because measurement systems were not available until recently to record data over a sufficiently long period of time and in enough detail to provide input

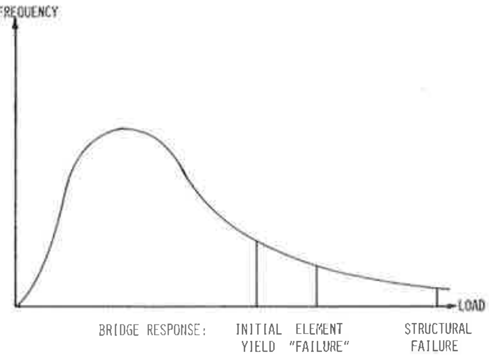


Figure 1. Frequency distribution of maximum lifetime loads illustrating range of bridge responses.

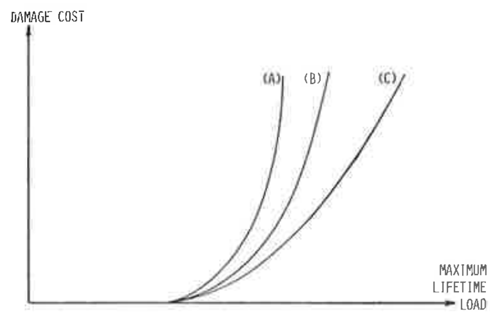


Figure 2. Damage cost versus load level for idealized bridge responses (deterministic case).

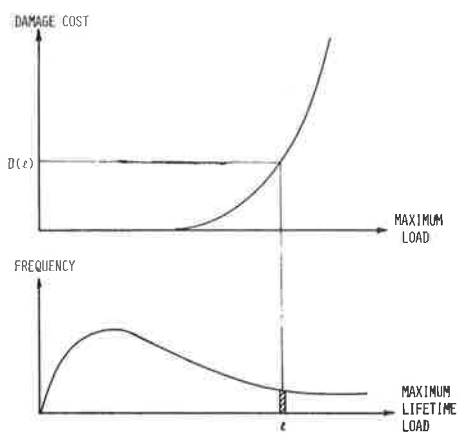


Figure 3. Expected damage cost obtained by integrating over maximum lifetime load probability distribution.

to simulation or prediction models (14, 35). All the various load-prediction studies cited above make reference to the lack of accuracy in the high-load portion of the truck-weight histogram, which controls the expected maximum load.

These factors suggest that it will be difficult to extend probabilistic methods to deriving load factors. It is possible, however, by broadening the limit-state definition to cover the range of structural responses and devising strategies to produce a probabilistic format. The following section outlines some of the considerations involved in this analysis.

STRATEGIES FOR HIGHWAY LOAD SPECIFICATIONS

In contrast to other probabilistic applications that deal primarily with natural (environmental) loads, bridge loads are affected by users and highway authorities. Past experience indicates that the maximum lifetime load will depend on increases in both the legal limit and the extralegal vehicles operating above the authorized limits. Code writers, and by implication designers, should incorporate these considerations into a design-load strategy.

It is still a formidable task, even without evolving loads to derive a loading based on the maximum occurrence out of some 50-100 million independent loadings. The solution can be made plausible if the design load is, in fact, intended to provide a reference point on what is actually a continuum of loads. This implies that there is not a unique failure load even in a limit-state or ultimate-strength format. Thus, the load factor can be chosen or calibrated for strategic and computational reasons. It need not necessarily be statistically based, as, say, the load with a 100-year return period as in environmental load phenomena.

The frequency diagram of maximum loading in Figure 1 illustrates the possible behavior responses including yielding, element failure, and finally damage and collapse. The function of the specified load is to simplify the design process and to minimize damage over the full range of behavior responses. Factored loads in general are not collapse loads but rather are based on element behavior, which is usually the load at which damage begins to occur. The expected damage can only be estimated by incorporating the system behavior of the structure to give the likely bridge damage in response to a range of overloads. This will involve qualities such as ductility, redundancy, and nonlinear behavior.

The models of damage curves illustrated in Figure 2 show the increase in damage cost (repair, public inconvenience, etc.) versus the level of loading for idealized cases ranging from little ductility and redundancy (curve A), to medium redundancy (curve B), to large degrees of redundancy (curve C). The damage curves are given as deterministic relationships between damage D and load occurrence L. By representing the uncertainty in the load occurrence as illustrated in Figure 3, where the shaded area is $f_L(\ell)d\ell$, the mean damage cost $\bar{D}$ is found by integrating over the load random variable L as

$$\bar{D} = \int D(\ell) f_L(\ell) d\ell \qquad (17)$$

where $D(\ell)$ is the expected damage due to a load ℓ and $f_L(\ell)$ is the probability function of load L.

Equation 17 can be further expanded to represent uncertainties in bridge behavior and damage cost by another level of integration on the conditional distribution of damage given load $f_D(d|\ell)$ (see Figure 4). This gives the expected damage cost $\bar{D}$ as

$$\bar{D} = \int_L [\int_D f_D(d|\ell) dd] f_L(\ell) d\ell \qquad (18)$$

By representing the load-factor magnitude as a decision variable, we can plot its influence on estimated total cost $\bar{C}$, which is made up of two components, initial cost (C_I) and damage cost ($\bar{D}$). Equation 18 assumes that the load distribution itself is not correlated with the maximum load because of legal limits or enforcement. These points are discussed below.

Figure 4. Expected damage cost obtained by integrating over maximum-lifetime-load probability distribution and conditional-damage probability distribution.

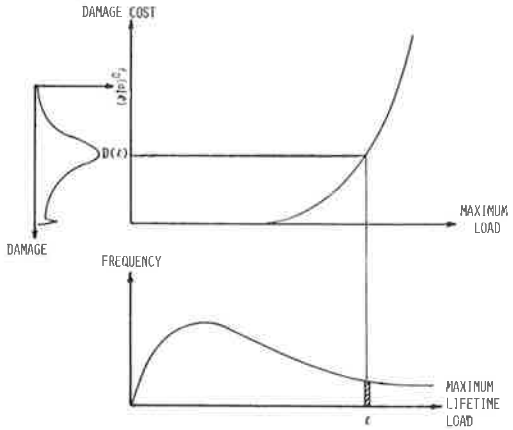
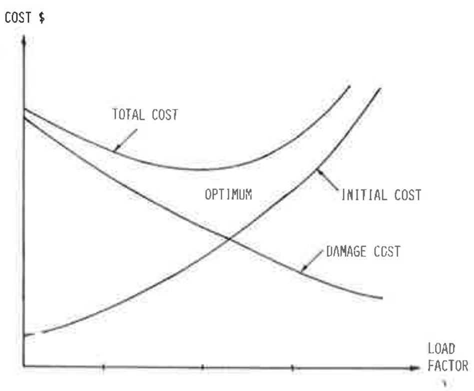


Figure 5. Initial cost, damage cost, and total cost plotted against load factor.



For illustration, all costs are also assumed to be brought to the same reference time to avoid introducing interest rates that must also be entered in the cost analysis. The total estimated cost can then be written as

$$\bar{C}(\gamma) = C_I(\gamma) + \bar{D}(\gamma) \tag{19}$$

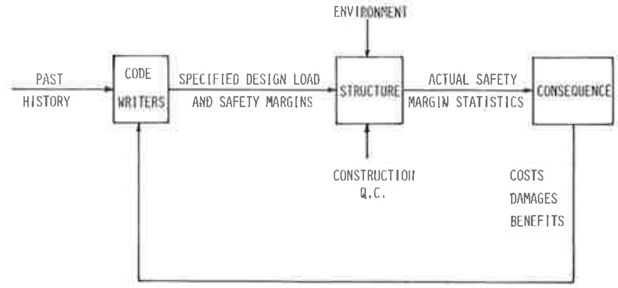
As the load factor γ increases, the initial cost increases and the damage cost decreases. An optimum load factor can be found by plotting these curves as illustrated in Figure 5. The optimum load factor occurs when the slope of initial cost equals the negative slope of the damage cost.

An optimization approach to deriving load factors is obviously a higher-level reliability format than the safety-index representation (31). It also has the advantage of rationally defending legal load limits as the optimum utilization of available resources.

These analyses also affect the design concept because they show the importance that redundancy, or parallel load paths, has for overall safety and cost. For structures that have large capabilities for load redistribution following an element failure, the damage curve increases only gradually and the bridge can accommodate overload with little damage cost. If the structure has little redundancy, then major damage cost is realized for loads only slightly greater than the element-failure load.

This discussion shows that the solution to load

Figure 6. Flow of information in code calibration process with environmental loads (invariant load process).



specification in a probabilistic context also involves the structure's behavior and particularly its total system performance. The alternative, which is to model current load statistics as the guide to future bridge performance, will usually prove erroneous. For example, a fatigue-damage assessment by Moses and Garson (18) showed that the contribution of illegal overloads is substantial. Low truck volumes, say around 40 trucks/h, overloaded only 2 percent above the legal limit contributed to a significant increase in fatigue damage, which was comparable to the effect of multiple-crossing superpositions. At a rate of 10 percent overload, even for high truck volumes, the contribution of overloads to fatigue damage greatly exceeded the multiple-crossing effect and dominated the entire fatigue calculation.

The specification of bridge design loads is an interplay of economic and engineering considerations. Bridges can be built to safely carry any conceivable design load, and instances have been reported of bridges and even entire highway networks being designed for special truck loads weighing 450 000 kg (1 000 000 lb).

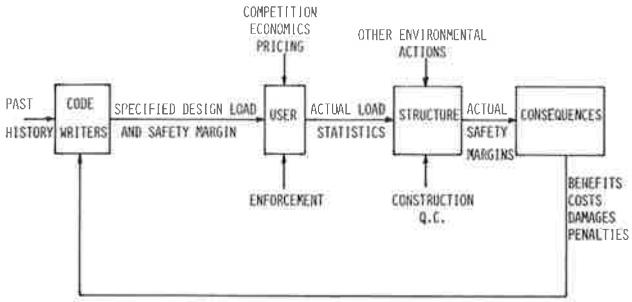
The problem, of course, is to control and integrate design-load limits with the existing network of highways and bridge structures. There is feedback in this model between specified loads and the corresponding maximum observed loads. Safety margins are known also to lobby groups pressing for increased legal load limits who can claim wide safety margins as an argument for increasing loads (36).

In addition, the efficiency of weight enforcement should be incorporated in the damage-cost model outlined above. For example, what is the probability of obtaining a load equal to the collapse load, given that damage evidence indicates a load exceeding yield has already been reached? In other words, is it possible for enforcement to limit the extreme tail of the load distribution? In such cases, structures with fuses that exhibit any damaging loads are desirable. Alternatively, more field instrumentation to continuously monitor behavior of one type developed by Baldwin (35) would be useful.

Figure 6 is a schematic of a calibration process in the framework established by Siu, Parimi, and Lind (7) and Galambos (34) that allows experience with existing structures to influence the reliability-based factors. The flow of information is from field experience back to the code writers, assuming unvarying load descriptions (usually environmental in nature). The process, if continued, would allow structural design to proceed along the hierarchy discussed by Esteva and Rosenblueth (31) that ranges from partial factors to calibrated safety indices, design for risk, and finally optimization of risk.

The highway bridge model is more complicated, as illustrated in Figure 7, where the load magnitudes are affected by the code decisions and the specification of

Figure 7. Flow of information in code calibration process with highway vehicle loads (human-controlled load process).



safety margins. For example, a long period that passes without loads causing bridge damage would encourage increases in legal limits or perhaps reduced enforcement.

Probability distributions of future loads that are functions of the specified load, economics of enforcement, and the fuse effect of the bridge design must be derived. As an example, one study (37) showed that the proportion of overweight axles was quite uniform in various jurisdictions and independent of the axle-load limit. Obviously, economic laws as well as structural behavior must enter the code writers' thinking.

CONCLUSIONS

1. Probabilistic methods can describe vehicle loads in the context of load factor, load and resistance factor, or other safety-index formats. Load factors can be derived as functions of mean and variance of load distribution for both repetitive fatigue spectra and maximum lifetime loading. Railway bridge design and rating appears to have immediate and direct application because future loads can be controlled by railroad operators.

2. Modeling highway load distributions has been accomplished with available vehicle weight data, preferably with computer simulation or convolution techniques. Examples of code-oriented research for both fatigue and strength (maximum lifetime load) have been described.

3. The principal limitation in probabilistic load description for highway applications is the relatively sparse data on extreme vehicle loads and combinations of vehicles (spacing models). Since highway loads are also evolving, past experience is not a direct calibration guide for load factors as it is for other structural applications.

4. A system-reliability model is proposed for incorporating probabilistic concepts in highway bridge design. In a manner analogous to that of a fracture-control plan, it is proposed that bridges be studied in the range beyond element behavior. Code groups and researchers should study various categories of bridge configurations to produce curves for damage cost versus load. These curves can then be operated on with probabilistic descriptions of load and resistance uncertainties to calibrate optimum load factors. Future changes in loads would be partially absorbed in the damage range between system failure and element failure. Structures designed with adequate ductility and redundancy would have lower load factors than do bridges with small amounts of such reserve strength. An example of a reliability analysis carried to the ultimate load level was recently published for offshore platform structures (38).

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Discussion

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Some of the concepts and findings introduced in this paper regarding bridge design loads, response of bridges to various loading conditions (especially when the load levels are high enough to introduce material nonlinearities), and the load level versus expected damage to the superstructure could be further amplified by the findings of recently completed research programs (39).

As Moses indicates, the strength of the bridge superstructure and its components can only be correctly assessed by considering the full superstructure as a single entity. Any formulation based on the analysis of bridge components as separate units will lead to erroneous results (39). The source of the discrepancies stems from the interaction between different components of the superstructure, which is not normally taken into account in the design phase (1). The problems will be further compounded if one must determine the post-elastic strength of the superstructure, either up to the attainment of the serviceability limits or up to the failure limits. At this load range, parts of the superstructure will exhibit varying degrees of material or even geometrical nonlinearities, which will cause the redistribution of the live load (41). Furthermore, the employment of the distribution factors will also lead to incorrect results if the superstructure is exhibiting appreciable inelastic response, regardless of whether it is recoverable or not (39, 40). It can then be concluded that any accurate prediction of the response of the superstructure beyond the linear elastic limit must resort to analysis schemes, such as the finite-element method, that contain provisions for the overall analysis of the structural system and will contain provisions for the nonlinearities.

An analysis system for beam-slab bridges with reinforced concrete decks and reinforced or prestressed concrete T- or I-beams has already been developed (41). The prediction of the static response of the superstructures beyond elastic limits by using component analysis as opposed to total analysis indicates that results based on component-by-component analysis yield results that increase in inaccuracy as the serviceability limits and especially failure limits are approached.

The paper indicates the need for the development of the curves for replacement cost as a function of maximum lifetime load. The establishment of cost-related values tends to depend on the location of the bridge and the material and labor costs associated with the replacement program. However, it is possible to establish deterministic relations between the projected overload configurations and the type and amount of damage that the bridge superstructure will sustain. A parametric investigation into this for prestressed concrete I-beam bridges has been completed (42).

Similar developmental research for steel-girder bridges is under way. Parametric investigations of bridge superstructures subjected to predetermined overload configurations will permit establishment of a sufficient data base that relates load configurations to the type of damage that the superstructure will exhibit. This, in due course, can and will provide the necessary information for the establishment of the load levels.

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Importance of Redundancy in Bridge-Fracture Control

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Because of component redundancy, riveted structures have tended to be fail-safe. It has been far less important to be aware of the limits of fatigue and brittle fracture in riveted structures than in welded structures, which are generally not component fail-safe. In the change from riveted to welded-plate girders, the safety factor protecting against brittle fracture in nonredundant load-path structures has weakened. The inherent crack stoppers at interfaces between components of riveted structures do not exist in structures that are welded or repaired by welding. Designers must therefore design, fabricators must produce, and inspectors must examine relatively crack-free structures and ensure that they will not develop large cracks during their service lives. This safe-life approach is an absolute requirement for nonredundant load-path structures. Several examples of cracked structures that have not collapsed because of redundancy are given, and the effect of welded repairs is discussed. The paper illustrates the redundancy of several simple trusses with a discussion of bridge fires. Strict application of these guidelines will force many designers to change to redundant load-path or component-redundant structures (e.g., bolted) in many instances, particularly in the short-span range, as alternatives to the additional material that may be required to avoid fracture.

Fatigue and fracture are apparently far more serious problems in welded structures (1) than in riveted structures. Part of this is because, during the long experience with riveted structures, most of the really bad details were eliminated; part is also because riveted structures have an inherent component redundancy and somewhat lower rigidity.

Just after the turn of the century use of redundant members was frowned on. Waddell (2) pointed out that the resulting ambiguity of stress distribution could lead to insufficiently designed connections or to an error in following a load to its conclusion. He also emphasized that, in checking a structure, one must follow each stress given on the stress diagram, from its point of application on one main member until it is transferred completely either to other main members or to the substructure, and see that each detail by which it travels has sufficient strength to resist the stress that it carries.

Clearly it was not possible to apply these principles to a structure in which the designer had no idea of the load path.

REDUNDANCY

Against fracture, however, riveted structures were at least internally member redundant in that most members were built up of several components (Figure 1). This component redundancy comes about because cracks do not jump from piece to piece.

The chord member of the truss shown in Figure 2 cracked on only one side of the member and carried rail traffic for some time before detection and repair. In a welded-box member the crack would have propagated all around (Figure 3).

In spite of the above principles, most trusses were multi-load-path structures. The truss in Figure 4 had its bottom chord completely severed and yet remained standing because of the alternate load paths provided by the bracing, floor system, etc. Unfortunately, the truss shown in Figure 5 did not follow Waddell's rules for adequate bracing and did not remain standing when one of its diagonals was severed by a shifting load. It had insufficient redundancy.

To demonstrate clearly that redundancy did exist in structures designed by these principles it is only necessary to examine a few bridge fires and their resulting locked-in residual stresses.

One of our trusses was subjected to a deck fire. On cooling, a gun-like explosion indicated a crack in the bottom chord (Figure 6). In a simple truss that had no other mechanism for carrying load, this fracture would have caused collapse. Over the next three weeks early spring temperature cycles caused several more of these rapid fractures. There was no live load and the dead-load stresses were rather low, so the driving force must have been locked-in residuals from the fire coupled with the relatively small stresses from temperature varia-