Application of Load Spectra to Bridge Rating

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ABSTRACT

Important safety decisions are made each time a bridge is evaluated. Field inspections have concentrated on estimating deterioration and dimensions of load-carrying members. How to measure and use a load spectrum at the site is described. Information on truck loads, dynamic impact, and girder distribution can provide additional data for rating bridges. Five sites in Ohio are reported. In addition, almost 100 other instrumented bridges have been studied by a similar weigh-in-motion operation, which uses existing bridges to provide equivalent static weights of passing vehicles. Weight data are unbiased because the field operation is undetected by drivers. The measured bridge load spectra can replace conservative AASHTO rating recommendations for impact and girder distribution factors. In order to enhance this application a reliability or probabilistic approach incorporates the measured site load spectra in evaluating the bridge safety. Loading is modeled by random variables including truck weight, traffic volume (affecting multiple presence), axle spacings and loads, impact, girder distribution, and measured stresses. A load simulation forecasts the maximum response for periods corresponding to inspection intervals. The calculation incorporates uncertainties and provides a reliability measure for comparing bridge safety. Examples include ultimate strength and fatigue-limit states. Strategies are described for using the load spectra and the reliability model to develop load factors for rating, schedules for inspection intervals, posting control, and redundancy evaluation.

The evaluation or rating of existing bridges is a continuous activity for most bridge bureaus. Vital safety decisions must be made to repair, rehabilitate, post, close, or replace an existing bridge. Existing manuals provide inspection techniques and guidelines for rating. The field inspection establishes deterioration and dimensions of load-carrying members. The rating (strength) checks generally follow procedures similar to AASHTO design including specified design loads, girder distribution and impact factors, and allowable stresses. Some flexibility in choosing safety factors is usually available.

Recent proposals for rating modifications include load-factor design and reliability-based load and resistance safety factors (1). One goal is to modify rating values if additional field inspection effort or analysis and load response are carried out (2).

BRIDGE LOADING SPECTRA

Ideally, the bridge rating engineer is in a better position than a designer to establish more precisely both the loading spectrum and the capacity for an existing structure. The uncertainties that should affect the safety factors are quite different for an existing bridge from those needed for a design that is not yet built. The acquisition of dimensions and material properties is routine and will not be described here. However, the acquisition of load and bridge response data is not routine, even though hundreds of bridges have been tested in many countries. One reason for its limited use in rating may be the need for a simplified measurement system. Bridge tests are costly and often use specialized equipment and processing programs. Equipment and test procedures must be available for routinely measuring load spectra. A second difficulty is the incorporation of a measured load spectrum into the formulation of rating factors.

In this paper some field methods for routinely acquiring bridge load spectra and response statistics and a probabilistic model for applying this information will be described. The field measurement system for obtaining load spectra is an extension of the weigh-in-motion (WIM) concept developed at Case Western Reserve University for the Ohio Department of Transportation (ODOT) and FHWA (3). It was originally developed to provide truck weight and traffic statistics. A recent test program extended the methodology to obtain bridge performance data also (4). This information on truck loads, bridge girder stresses, and dynamic response can provide valuable data for evaluating an existing bridge.

In order to utilize this data base of acquired load spectra a reliability-based formulation is described. It can calibrate appropriate load factors in conjunction with predictions of the maximum expected truck loading and can even account for parallel-redundant load paths. The reliability model also incorporates the measured statistics of girder distribution and impact. Risk assessments of ultimate strength and fatigue lives are given. Strategies based on the acquired site-specific load spectra are discussed for inspection, rehabilitation, and permit control.

LOADING ANALYSIS

For most short- and medium-span bridges, the critical loading is self-weight and heavy truck traffic. Self-weight can be estimated during inspection from cores and recorded dimensions. The repetitive heavy vehicle loads may cause fatigue cracks, instability, permanent displacement, or collapse.

Each live-load event depends on truck weight and axle loads and intervals between closely spaced vehicles (headways). In a critical component, stresses also depend on load distribution and bridge dynamics, which for the design were estimated from simplified models. Current load specifications also reflect the truck traffic in existence many decades ago. Changes in truck traffic, including heavier legal and permit vehicles and other trends, are important. Such changes are as follows:

1. Increased gross weights: unless accompanied by longer axle lengths, heavier vehicles induce greater longitudinal bending moments;
2. Influence of closely spaced axles: increased tandem and triaxial weight combinations significantly affect component stresses sensitive to concentrated wheel loads;
3. Traffic increases: the frequency of platoons

of closely spaced vehicles, superimposing their load effects, increases with higher volumes;

4. Enforcement: there is concern that citizen-band (CB) radio communication and by-pass options have decreased legal load enforcement; also, little is known about whether posting signs has any effect on restricting loads;

5. Maintenance: bridge load spectra measurements show that the major influence on dynamic response is roadway roughness (4,5); and

6. Bridge lives: it is evident that initial estimates of 40 to 70 years for bridge lives are often being surpassed.

Live-Load Variables

The random girder stress (S) caused by a truck movement across a bridge in a typical multistringer bridge may be written as

$$S = mWghI/S_x \qquad (1)$$

where

- W = the vehicle weight,
- m = a factor to convert gross weight to bending moment,
- g = girder distribution factor (stringer analysis),
- h = variable to account for influence of multiple vehicles on overloading (function of traffic volume),
- I = impact due to dynamic response, and
- S_x = girder section modulus.

In design or rating manuals the load (W) and moment factor (m) are specified by the recommended axle or lane loads. The analysis or load distribution to individual girders is also specified, for example, girder spacing divided by 5.5, for steel girders (6). These factors nominally assume some multiple-presence arrangement, represented here by h. The dynamic allowance is also specified and formulas are usually given for calculating effective section moduli. The prediction of the loading either for repeated spectra (fatigue) or maximum response (strength) must include the uncertainties in W, m, g, h, I, and S_x.

For a new design, uncertainty in W and m and the volume (affecting h and fagitue life) will be large, especially when projected over long periods. Similarly the analysis uncertainties g, I, and S_x will be significant even with accurate finite-element analysis because of variations in stiffness factors, dimensions, and long-term changes.

In a bridge evaluation, there should be considerably lower levels of uncertainties if measurements of load spectra can be made. In addition, if the inspection or evaluation intervals are short (less than 5 years), the impact of uncertainty in future traffic projections should be minimized. A description of how the load spectra study can be performed is given later.

Reliability Modeling

A safety criterion is needed for evaluating existing bridges or designing new structures. Basing the safety criterion on the traditional allowable stress method may lead to inconsistent designs. A better approach has been shown to be load-factor design in which different load factors for dead and vehicle loading can account for respective levels of uncertainty (1,6). A rational safety goal is to keep the failure risk below some economically acceptable limit. The difficulty lies in calculating risks in the likely range of usefulness, typically less than 0.001 failure rate per year.

As a consequence a reliability model has been introduced in developing design codes in the United States and abroad for buildings, offshore structures, and so forth (7). It has also been adopted in Canada for bridge design (1). It is based on a nominal measure of reliability, namely, a safety index, which can be implemented without detailed probabilistic calculations. The safety index (often called beta) can be used in bridge rating for two major purposes: establishment of priorities for bridge rehabilitation based on safety measures and incorporation of past experience to establish target safety indices for rating limits.

A general model for reliability begins with a failure function (g) such that $g < 0$ implies failure. A simple case would be a structural element with strength R loaded with dead (D) and vehicle (L) loads. Thus,

$$g = R - D - L \qquad (2)$$

expresses the safety criterion. The safety index can then be expressed as follows:

$$\text{Safety index} = \text{mean of } g/\text{standard deviation of } g \qquad (3)$$

Accurate calculations of β have been developed that for many distribution functions provide a good agreement with risk determined from simulations. Risk is given as follows:

$$\text{Risk} = F(\beta) \qquad (4)$$

where F is the normal (Gaussian) distribution.

An excellent source for this material is the recent report on the formulation of the ANSI A-58 building code (7). It also contains a computer program for calculating β given an equation for g. An important application of β is the calibration of load factors (γ) and resistance safety factors (ϕ). A target β (typically in the range of 2 to 4) is determined from existing specifications and field performance. The calibration finds the safety factors in a load- and resistance-factor safety check similar to load-factor design:

$$\phi R > \gamma_D D + \gamma_L L \qquad (5)$$

γ's and ϕ's are found to provide the target β over a full range of applications. For example, an analysis of the current AASHTO code showed how load factors can be chosen to give more constant β's for different spans and ratios of dead to live load (8). The input in the β calculation is the means and variances of each of the load and resistance variables. Although data may be limited, a sensitivity study showed that the calibration exercise reduces the importance of small changes in the data base. This occurs when both the target β's and the safety factors use the same data base.

Subsequently, β's are calculated for the fatigue-limit states and the maximum-load-limit state.

FIELD MEASUREMENT OF LOAD SPECTRA

The most important ingredient in the load model is accurate truck weight statistics. Avoidance of static scales is well recognized and by-pass routes make such scales ineffective for obtaining accurate highway weight statistics. For several years there has been worldwide interest in producing an undetectable system for automatically weighing trucks moving at normal highway speeds. A variety of pavement insert scales have been tested. These flexible plates respond to vertical forces and are calibrated to give histograms of recorded wheel loads. The problems encountered are due to scale flexibility and the bounce when a massive flexible vehicle moves on a rough pavement at high speeds. The vehicle is typically on the scale for only a portion of its natural period, and large systematic errors may occur because of force oscillation. As a consequence, pavement scales are more accurate for low-speed sorting at busy weigh stations.

Recently the authors and their colleagues at Case extended a system of bridge measurements (5) used to obtain strain histories to also obtain truck weight information. The weighing system has reached the stage of relatively routine operation by ODOT (3,4), FHWA (9), and other groups to monitor truck weights. Thus far, more than 100 sites have been surveyed.

Briefly, the WIM system uses existing bridges as equivalent static scales. Trucks move at normal speeds and drivers cannot detect the weighing operations. Vehicle speeds and dimensions are obtained via tapeswitches bonded to the roadway (Figure 1). Bridge girder response comes from reusable strain transducers clamped to steel flanges or bolted to concrete beams. The girder influence line provides a simulated strain record. The vehicle axle weights are obtained by automatically matching the measured and simulated strains (10). The data recording, monitoring, and weight calculation are done in real time on a minicomputer in an instrument van usually parked beneath the bridge. A known calibration truck is used to establish a relationship between strains and truck weight.

Sites monitored by this procedure have included single-span and continuous steel girders and reinforced and prestressed-concrete beams. The accuracy of the WIM weighing has been verified in several studies by comparing it with static weighings (3,9). It is important for planning that the weight predictions be unbiased. The WIM surveys have provided general weight trends, which, however, are still limited for bridge load and fatigue-spectra modeling.

In a recent modification to the system, strain and traffic data were taken on a continuous basis (4). This provides a total picture of truck traffic including weights, lane occupancy, headways in each lane, maximum stresses, girder distributions, impact, and so forth. Field operations were performed during the summer of 1982 at five sites in northeast Ohio including four steel and one concrete-beam bridge. An example of a recorded event is given in Figure 2. The event shown is two trucks moving side by side. The processing of the strain record gave a 68.3-kip vehicle in the driving lane and a 31.4-kip vehicle in the passing lane with axle weights as shown. This processing required influence coefficients for each lane and girder, which are obtained with the calibration vehicle run at normal speeds in each lane position. One influence example is shown in Figure 3.

Some 8-16 hr of continuous recording were typically obtained at each of the sites investigated. On a routine basis it is expected that a site could be monitored in about 2 days, including set-up, calibration, data acquisition, and processing. It should be noted that the truck weight and traffic data are applicable to any bridge along the same highway, whereas the bridge response parameters (impact, girder stresses) apply only to the structure being studied. Thus, either a rehabilitation or a replacement structure would also benefit from the load spectrum.

To illustrate the acquisition of a load spectrum, a typical site study is outlined. The application to reliability models for rating, fatigue, and load prediction is given in the next section. Table 1 contains the truck weight distribution measured by the WIM system at a site in Ohio on I-90. It shows average gross weight, standard deviation, and average axle distributions for the most common truck categories. An example of a maximum stress distribution is given in Figure 4, which illustrates the low stress levels (less than 6 ksi) observed in most bridge studies. Table 2 shows average measured girder distribution factors for the several steel girder sites; the corresponding AASHTO values are included. Note that when trucks are occupying both lanes, the AASHTO distribution generally is conservative.

Dynamic factors in Table 3 were calculated from examination of the strain oscillation. This has not yet been automated because the dynamic oscillation can be confused with static strain variations caused by axle spacings. It is inaccurate to use spectral

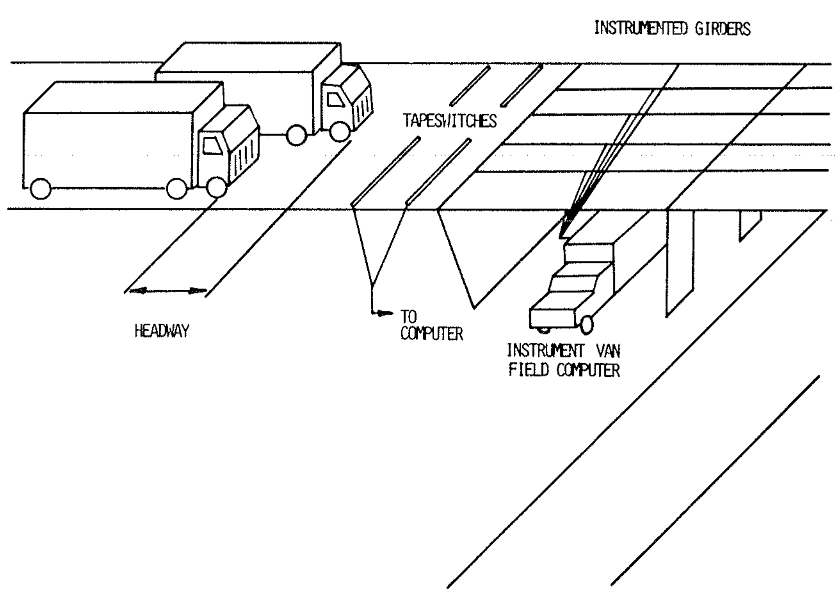

FIGURE 1 Typical WIM installation.

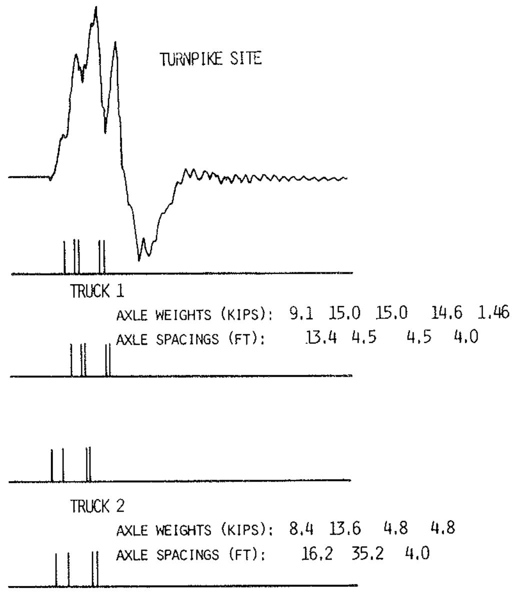

FIGURE 2 Sample record of two side-by-side trucks.

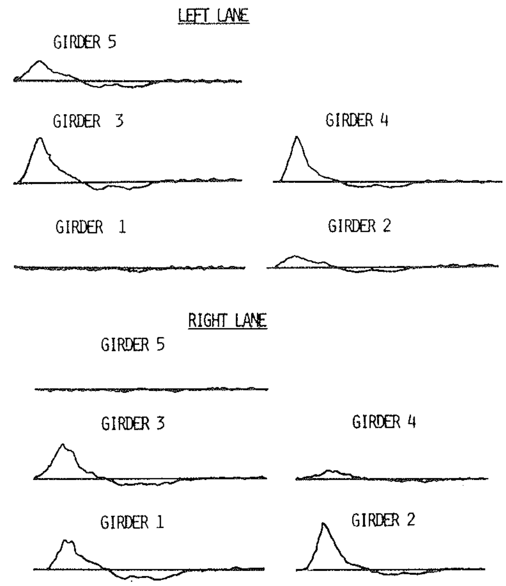

FIGURE 3 Influence lines for a five-girder bridge.

TABLE 1 Truck Statistics Obtained by WIM System (Ohio, I-90)

Category	No. of Vehicles	Percentage of Total	Gross Weight (kips)		Axle Weight Distribution (%)		
			Avg	SD	Front	Drive	Rear
Two-axle single							
Lane 1	63	12	15.8	5.94	33.57		66.44
Lane 2	9	2	15.8	4.14	30.34		69.62
Three-axle single							
Lane 1	29	5	27.9	6.74	30.09		69.46
Lane 2	5	1	25.6	5.09	36.12		63.85
Four-axle semitrailer							
Lane 1	45	9	32.4	12.12	17.89	38.25	43.80
Lane 2	4	1	32.9	5.06	23.87	35.07	41.03
Five-axle semitrailer							
Lane 1	217	42	50.8	17.73	13.86	48.08	38.03
Lane 2	52	10	48.8	18.49	14.04	48.43	37.49
Five-axle split							
Lane 1	37	7	53.87	17.55	11.93	47.54	40.50
Lane 2	10	2	39.98	15.50	19.86	45.72	34.38

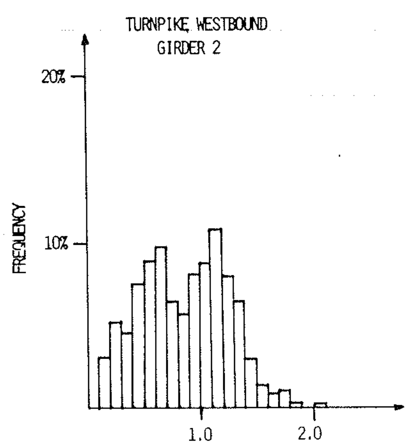

FIGURE 4 Histogram of maximum stress distribution.

TABLE 2 Average Girder Distribution Factors

Site	Percentage of Total Stress by Girder						AASHTO Value (6) (%)
	1	2	3	4	5	6	
I-90							
Case 1	6.4	19.1	40.6	25.9	7.1	0.8	
Case 2	0.2	7.4	19.2	29.9	29.6	13.6	
Case 3	3.3	13.3	29.9	27.9	18.4	7.2	36
I-71							
Case 1	7.6	27.2	33.9	20.5	9.5	1.5	
Case 2	−0.3	14.0	22.5	30.1	23.0	10.6	
Case 3	3.7	20.6	28.2	25.3	16.3	6.1	36
I-80 westbound							
Case 1	18.2	37.9	34.0	9.7	0.1	NA	
Case 2	−0.1	9.9	34.2	37.2	18.9	NA	
Case 3	9.1	23.9	34.1	23.5	9.5	NA	33
I-80 eastbound							
Case 1	23.7	37.9	25.9	11.8	0.6	NA	
Case 2	0.3	11.0	27.2	42.9	18.6	NA	
Case 3	12.0	24.5	26.6	27.4	9.6	NA	33

Note: Case 1: truck in right lane (measured); case 2: truck in left lane (measured); case 3: side-by-side trucks of same weight (hypothetical). NA = not applicable.

TABLE 3 Dynamic Factors for Typical Truck Records (Ohio, I-90)

Record	Impact[a] (%)	Record	Impact[a] (%)
1	15	13	26
3	13	17	26
6	11	18	23
8	17	19	14
10	20	20	19
12	23		

[a]Measured on most heavily loaded girder.

analysis for finding dynamic response for bridges with spans less than 125 ft. Figures 5 and 6 show headway spacings between moving trucks in the same lane or moving in different lanes. This gives the data for constructing the load superposition model or the headway variable (h) defined earlier. Data on section modulus S_x can also be inferred by taking the bending moments and dividing by the girder stresses. In the instances studied, such data might be misleading because the bridges were designed to be noncomposite but obviously exhibited considerable composite action. More detailed study of the variable S_x is still needed.

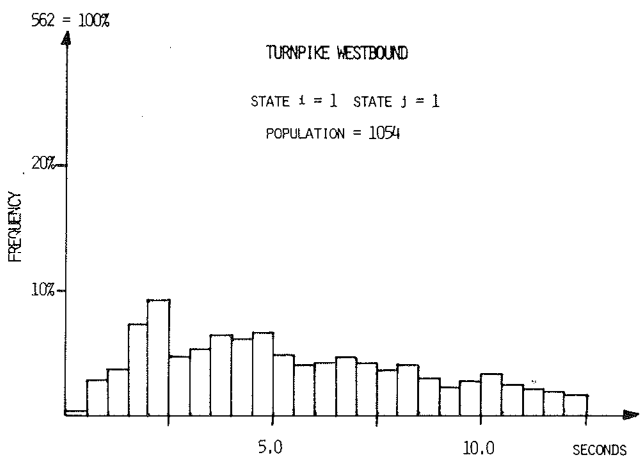

FIGURE 5 Histogram of headway for trucks in right lane.

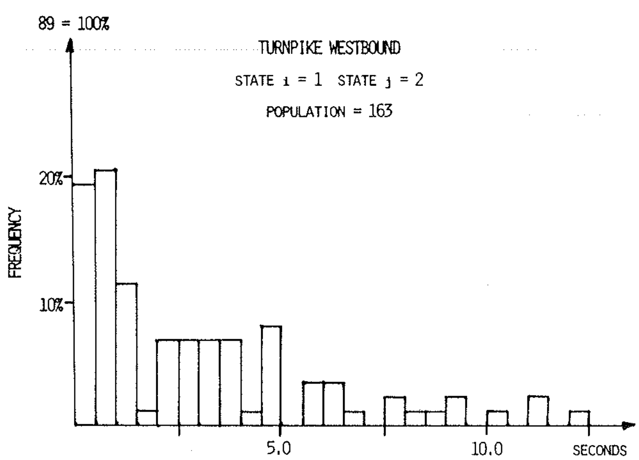

FIGURE 6 Histogram of headway for trucks in left lane approaching trucks in right lane.

RELIABILITY ASSESSMENTS

The previous section showed how a load spectrum data base could be acquired. This section provides application of the data to load forecasting and calculation of safety indices for ultimate strength (component and system) and fatigue.

Load Forecasting

Load spectrum data have been typically taken for 1- or 2-day periods at each site. Adjustments for daily, weekly, and seasonal variations can be made by using more extensive weight survey information gathered for pavement, enforcement, and other planning purposes. The first step in bridge reliability modeling is to forecast maximum bending moments. These depend on the truck weight distribution, axle spacing and axle weight distribution (variable m given previously), and truck volume (which affects headway). As an illustration the load data taken at an I-90 (Ohio) site are used to forecast distribution of maximum bending moment for a 100-ft simple supported span. Several types of load-modeling programs have been used in such forecasting, including simulation, Markov renewal models, and simplified approximations, all of which are in general agreement (11). Figure 7 shows a maximum moment distribution for a 10-year forecast by using five-axle vehicles with constant axle spacings and weight distribution. These forecasts, which ignore future growth in truck weights, are in the form of probability distributions. The mean and variance are the most important parameters needed in the failure function (Equation 1) to calculate β. This was done by using a level-2 reliability analysis (7). The data for dead load, live load, and strength are presented in Table 4. Using the failure function in Table 4 provided a β of 3.1. It should be noted that the live-load data are based only on a limited number of sites.

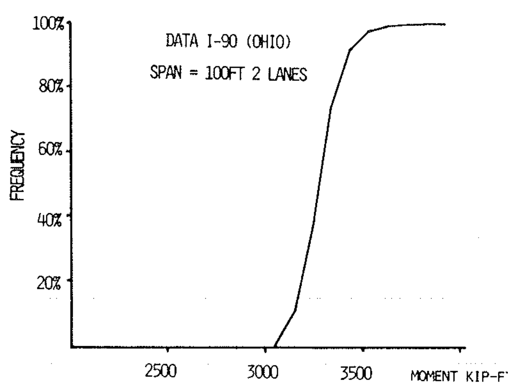

FIGURE 7 Distribution of maximum bending moment for a 10-year forecast.

For study of design safety and load factors, β's should be calculated for different span and support configurations. The aim is to obtain reasonably uniform β's over the range of code applicability. Because the current load specifications do not match measured load spectra, there will likely be scatter in β's. Taking an average of these β's gives an appropriate target and choosing the dead- and live-load factors (7,8) can also smooth out any variations in β.

TABLE 4 Data Base for β Calculations Without Measured Load Spectra at Site

Random Variable	Mean	Coefficient of Variation (%)	Comments
R	1.1 AASHTO (1,676.4 kip·ft)	15	Resistance: nominal AASHTO strength
D	1,673 kip·ft	10	Dead load[a]
m, W, h	4,450 kip·ft	20	Obtained from simulation of maximum moment by using measured truck data and average truck volume (V = 2,500/day)
g	0.3	11	Maximum girder distribution factor: based on 4 sites (4)
I	1.11	11	Impact factor: based on 10 sites (13)
Total live load	1,482 kip·ft	25	Calculated: live load = mWhgI

Note: Failure function = R − D − mWhgI (assume log normal distributions); 100-ft span (span length affects R, D, m).

[a]Based on data from Moses and Ghosn (8).

Rating

An important fact in rating is that the bridge is available for making observations relating to both capacity (deterioration) and the load spectrum. The rating should be viewed as part of an important control process in producing acceptable safety. A model of demand (load) and capacity (resistance) for a bridge similar to the fundamental reliability model may appear as shown in Figure 8. The load and strength frequency distributions for the as-built conditions will show little overlap, indicating high reliability. Over time, the strength deteriorates and the loads generally increase. If nothing is done to rehabilitate any damage or control the loads, the risk may increase to unacceptable levels. Inspection is part of this control process.

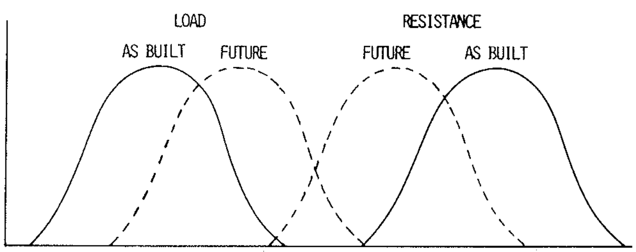

FIGURE 8 Reliability model for bridge rating.

To illustrate, consider the load model described previously. Figure 9 shows a simulated distribution of the maximum bending moments for periods of 1 day, 1 month, 1 year, and 5 years. The increasing load suggests that the inspection interval is important.

In the absence of a measured load spectrum, the design specification must be used to calculate design moments. These values will have greater uncertainty than a measured load spectrum determined at the site. For example, the overall live-load coefficient of variation is 25 percent without a measured spectrum, which compares with 15 percent in Table 5 when measurements have been taken. Note that only a few sites have yet been monitored, so the data base in Tables 4 and 5 must be viewed as still tentative.

As one application of this data, Figure 10 compares β's for different estimates of section deterioration for the case of an available load spectrum with the case in which measurements are not made. The higher β's reflect the lower uncertainties with measured load spectrum. In fact, another advantage of the measured spectrum is the identification of the mean load, which may differ significantly from the specification loads. For the same

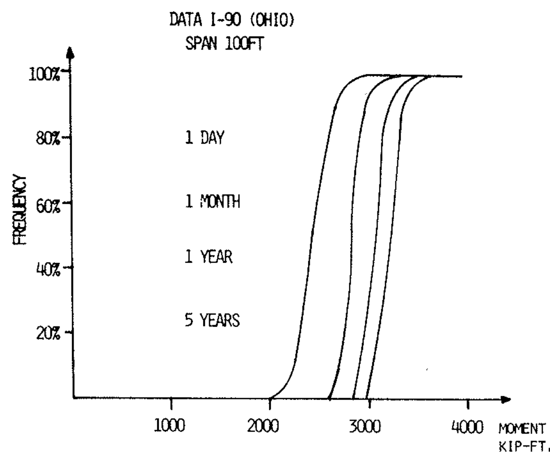

FIGURE 9 Distribution of maximum bending moment for different rating periods.

TABLE 5 Data Base for β Calculations with Measured Load Spectra (Ohio, I-90)

Random Variable	Mean	Coefficient of Variation (%)	Comments
S_x	404.8 in.^{3}		Section modulus: from site plans[a]
F_y	40 ksi	10	Yield stress (7)[a]
D_{et}	0.8	15	Deterioration factor: mean assumed from inspection[a]
D	186 kip·ft	10	Dead load: estimated from site plans
m, W, h	1,092 kip·ft	12	Obtained from simulation based on I-90 volume
g	0.30	8	Measured maximum girder distribution factor
I	1.2	11	Measured impact

Note: Failure function = $S_x F_y D_{et}$ − D − mWhgI (assume log normal distribution); 40-ft span, two lanes.

[a]$R = F_y \cdot S_x \cdot D_{et}$.

parametric data in Table 5, Figure 11 shows reduction in β with the increase of the mean value of the load spectrum. Figures 12 and 13 show the effect on β with measured impact and girder distributions as compared with using AASHTO specification values.

Redundancy

It is generally recognized that redundant or parallel load paths are necessary in case of accidental loadings or component failures (caused by fatigue and brittle or even ductile behavior). It is pos-

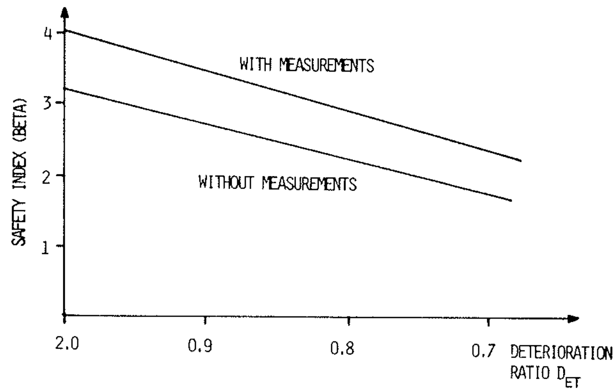

FIGURE 10 Beta for different estimates of deterioration (40-ft span).

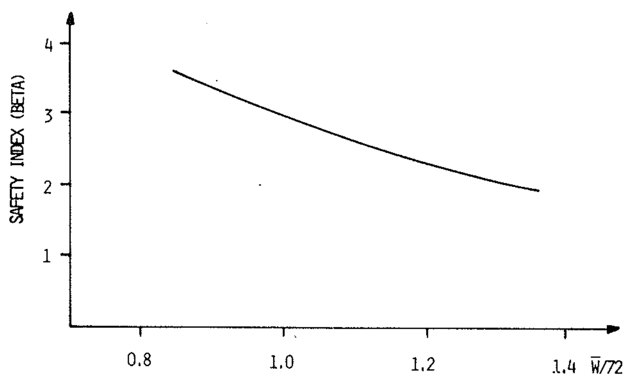

FIGURE 11 Beta for different values of mean load (40-ft span).

sible to compare, for example, a two-girder with a five- parallel-girder system, as shown in Figure 14. If the first yield capacity is the same for both systems, the five-girder system has both a higher collapse load and greater reserve capacity if one of the girders should fail. Different component failure sequences or failure trees have been modeled in a reliability framework and are reported elsewhere (11,12). A damage index (similar to β) is introduced to integrate the consequences of load occurrences beyond the initial component failure. Redundant systems will have lower expected damage indices than statically determinate structures. These damage indices may also be included in a rating strategy, but further work is needed to make the system methodology easy to apply.

Fatigue

Fatigue checks are not normally part of a bridge evaluation, because it would appear imprudent to recommend precautions such as posting based only on a calculation and no observation of cracks. Laboratory tests of similar specimens often show variations with orders of magnitude in fatigue life. Nevertheless, a fatigue check may often identify critical components for detailed field inspection and perhaps also schedule inspection intervals. A load spectrum can also be introduced in the risk assessment for fatigue damage.

The fatigue of steel bridge members is determined by an averaging process. Each vehicle crossing at time t contributes to the cumulative damage [D(t)]. The failure function may be written as follows:

$$g = D_f - D(t) \tag{6}$$

D_f is the damage at failure, which should have a mean of 1.0 according to Miner's linear cumulative damage rule. Summing over the frequency histogram of stress range cycles (S_i) gives (13)

$$D(t) = (Vt/c)\, \Sigma\, S_i^3\, f(S_i) \tag{7}$$

where V is the truck volume. The cubic term derives

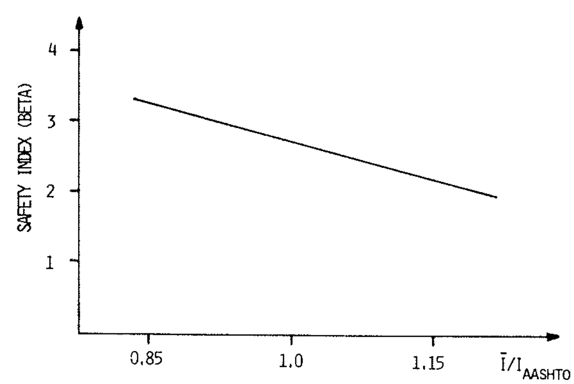

FIGURE 12 Beta for different values of mean impact (40-ft span).

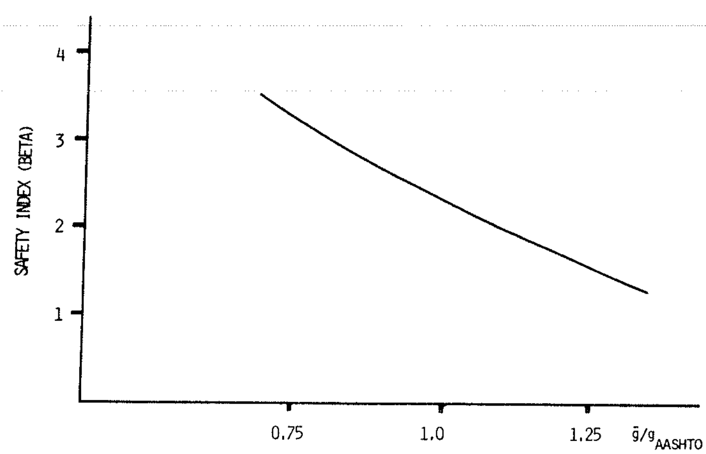

FIGURE 13 Beta for different values of girder distribution factor (40-ft span).

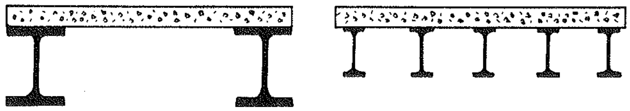

FIGURE 14 Example of bridge configurations.

from the slope of typical welded attachment S-N curves, whereas the intercept c depends on the fatigue category. The stress range (S_i) may be replaced by respective random variables by using Equation 1. The iterative safety index (7) program was run with typical AASHTO design factors and data (4,13). The results showed β's in the range of 2 to 3 for redundant cases and higher values for nonredundant cases. Proposals to modify the specification to produce a more uniform range of β's include the following:

1. Use a standard fatigue vehicle to calculate design moments rather than the AASHTO design with variable axle spacing (6).
2. Adjust the allowable stress ranges to produce similar β's for different attachments.
3. Allow more categories of heavy volumes instead of the single category of more than 2,000,000 because there are many roadways with an excess of 5,000 trucks per day. Beta is reduced with increased volume.
4. Allow more categories of heavy vehicle weight classification because some roadways are known to carry considerable numbers of heavy and overloaded vehicles. Beta decreases with increases in average vehicle weights.

It should be noted that the average weight that produces the same damage as the total population has been reported as 50 kips based on FHWA loadometer studies (14). Recent WIM studies, however, show regions with much heavier vehicles. These data are presented in Table 6 in terms of average vehicle weight to give the same overall damage intensity for a number of sites nationwide (9). This may show higher or lower β's than when the FHWA data base, which is averaged from many sites, is used. If the site is one with a high load spectrum, the β calculated may be low. This evaluation should be used to guide inspection frequency and the location of potential flaws.

TABLE 6 Fatigue Damage: Equivalent Truck Weight (9)

Site (9)	Equivalent Truck Weight (kips)	Site	Equivalent Truck Weight (kips)
CA-1	59.3	TX-1	54.3
CA-2	54.3	TX-2	58.3
CA-3	57.5	TX-3	57.1
CA-4	47.1	TX-4	54.3
CA-5	47.6	TX-5	52.2
CA-6	49.7	TX-6	58.3
GA-1	49.7	TX-7	67.1
GA-2	39.9	IL-1	69.8
GA-3	43.5	IL-2	48.2
ARK-1	53.5	IL-3	60.4
ARK-2	53.1	IL-4	52.2
ARK-3	44.7	NY-1	52.6
ARK-4	52.6	NY-2	44.7
ARK-5	51.2	NY-3	46.0
ARK-6	55.2	NY-4	56.4
OH-1	53.9		
OH-2	60.4		

CONCLUSIONS: STRATEGIES FOR BRIDGE EVALUATION

The routine acquisition of site-specific bridge load and response spectra has been outlined. In addition, a reliability assessment measure, namely, the safety index, could be used to rate the safety of a particular design. There are several possible applications of load spectra measurements that can lead to more efficient strategies for inspection, evaluation, and permit and load control.

Inspection

Funds are limited and reliability assessments may identify critical elements and assist in identifying inspection intervals. Fatigue-calculated β's may identify bridge locations where detailed crack investigation is warranted.

Posting

If low safety indices are found for maximum loading conditions, posting is warranted until repair or rehabilitation can be undertaken. WIM operations can assist in determining whether posting limits are obeyed.

Legal Load Limits

The impact of higher load limits will be reflected in the reliability model with lower safety indices.

Permit Loads

Reduced load factors may be warranted if loading is carefully controlled, as in the case of escorted permit vehicles. This can be reflected by reduced load uncertainties giving higher β's in the reliability model.

Enforcement

Evaluation of enforcement effectiveness is important in reducing load uncertainties. The impact of such enforcement becomes apparent in the calculated β values. The cost of increased enforcement should be balanced by the improvements in bridge safety.

Rating

Load and resistance factors in rating calculations need to be different from values used in design because of exposure period and available performance data (1). The existence of a measured load spectrum at a site should permit reduced load factors. Flexibility in incorporating the measurements in rating decisions will encourage bridge engineers to seek more field data to corroborate their calculations and enhance bridge safety.

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A Pragmatic Approach in Rating Highway Bridges

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ABSTRACT

A procedure is presented for rating highway bridges for regulation loads without causing yielding of the bridge materials. The procedure consists of three major parts: the measurement of regulation loads with a load measure, the yield capacity calculations of bridge members, and the ratings for various traffic conditions. The importance of accurate ratings, which will form the basis for making decisions pertaining to bridge upgrading and traffic control, is recognized. The results of the actual application of the procedure were found to be satisfactory in the strengthening programs of many existing bridges and in issuing overload permits. The procedure is considered to be simple, direct, and practical.

Highway bridges can have different ratings under different loading conditions. Because the actual traffic conditions are basically controlled by state regulations, it is logical to assume that regulation or legal loads resemble the various highway loadings. For upgrading an existing bridge economically, issuing overload permits, or posting for load limits, more reliable ratings for the regulation loads are desirable. Because any standard loading, such as HS20, as given in the AASHTO specifications (1) or a statistical truck model, is incapable of simulating all the load effects caused by the action of regulation loads of innumerable combinations of axle loads and spacings on various bridge members, it cannot yield reliable ratings. But by using a load measure, the actual traffic condition can be closely measured and thus more reliable ratings can be obtained.

A standard loading can easily be made into a load measure, for instance, by changing HS20 to HSW, where W is the variable combined weight on the first two axles. This simple transformation will make HS no longer a standard loading but rather a system of measurement. Like feet or meters for measuring lengths, the HS load measure may be used to obtain the load effects of various highway loadings. The proportional configuration of the HS load measure suggested is identical to that of the HS loading, which consists of either a three-axle truck or the corresponding lane loading. The only exception is that the spacing of the last two axles is fixed at 14 ft for the HS load measure.

The basic principle followed in this paper is to rate highway bridges for regulation loads without causing yielding of the bridge materials. Because