Evaluation of Steel Bridges Using In-Service Testing

MICHEL GHOSN, FRED MOSES, and JOHN GOBIESKI

ABSTRACT

Highway bridges often exhibit higher strengths than indicated by AASHTO rating procedures. This is because the code is inherently conservative and is intended to be applied to general situations. A more appropriate approach is to incorporate field observations in the rating process. A field measurements (weigh-in-motion) system is capable of providing all pertinent data on the loading and response of highway bridges. The data collected include measured stresses and girder distribution factors in addition to truck weights and volumes. The data are then incorporated in a working stress design rating or in a reliability-based safety evaluation of bridge members. Results from an example site indicate high safety levels despite the large numbers of permit vehicles allowed.

More than 100,000 bridges in the United States are reported to be structurally deficient (1). Many of these bridges, however, were designed and constructed in a manner that achieved greater strength than is recognized in conventional code rating provisions. Current evaluation and rating investigations emphasize bridge condition and member dimensions. Inspection methods rarely determine bridge loads or member performance under loading. Developments in weigh-in-motion technology, however, make it feasible to investigate existing bridges and provide more accurate site-specific load and response data for the evaluation process.

Bridge rating is a continuous and vital activity for most bridge bureaus. Safety and economic decisions must be made about each bridge: repair, rehabilitate, post, allow permits, close, or replace. Existing regulations prescribe inspection techniques and guidelines for evaluation. Field inspection establishes member properties, deterioration, and dimensions of load-carrying members and connections. Evaluation calculations generally follow AASHTO procedures (2). These are similar to the bridge design guidelines and specify loads, analysis (girder distribution), impact (dynamic amplification), and allowable stresses (3). The factors in the AASHTO design manual are necessarily conservative because they must apply to a variety of situations. Obviously, a specific bridge will have performance factors that are different from the ones cited in the code. For new construction, the additional cost associated with using conservative performance predictions is usually slight because adding capacity for new bridges will increase the overall construction cost by only a small amount. For in-service bridges, however, the cost of either adding capacity to the existing bridge or penalizing users with low posting or permit levels can be high.

The AASHTO bridge inspection manual, however, does permit wide latitude in selecting checking parameters if more data are available. There has been limited use of this flexibility in modifying evaluation parameters because the guidelines for incorporating any new data into the evaluation calculations are vague.

M.Ghosn, Department of Civil Engineering, The City College of New York, New York, N.Y. 10031. F. Moses and J. Gobieski, Department of Civil Engineering, Case Western Reserve University, Cleveland, Ohio 44106.

The objective of this paper is to provide a procedure that uses field measurements in connection with weigh-in-motion technology to assist in bridge evaluation and rating. Data were recorded on truck loads, dynamic impacts, girder distributions, and member stresses. These data are incorporated in an improved deterministic rating analysis and a proposed probabilistic approach for the evaluation of existing steel bridges.

EVALUATION PROCEDURES

Most states and bridge bureaus use rating procedures that generally follow AASHTO's guidelines. For example, the method followed by the Ohio Department of Transportation calculates the maximum moment at various points on the bridge due to a number of typical heavy trucks. This moment is multiplied by the impact factor for the location under consideration and the girder distribution factor, which depends on the bridge type and the girder spacings.

Usually, the available capacity for live load (member capacity minus dead load effect) is obtained by using the higher permitted operating stresses to reflect the uncertainties in strength and load associated with the rating period (usually 2 years) that are smaller than those associated with the design period.

Absent from the rating evaluation is any use of site-specific load and response information although it is obtainable by observing performance of an existing bridge. It should be noted that the AASHTO Maintenance and Inspection Manual (2) does allow considerable flexibility. For example, it states: "A higher safety factor for a bridge carrying a large volume of traffic may be desirable." Further, "impact may depend on deck roughness or approach." The manual also mentions consideration of the probability of closely spaced heavy trucks. These guidelines, however, are rarely used because the acquisition of relevant load data has, until recently, been difficult. Instead, most rating is done with prescribed factors that govern all bridges equally. For example, the same allowable stresses and nominal loading are used for heavily traveled Interstates and for rural roads with little traffic.

The following considerations, compared with the parameters in the AASHTO manual, may govern the true loading and response situation.

1. The girder distribution factor in AASHTO is

generally conservative although the margin on the safe side can vary considerably. It may depend on lane position relative to girder and curb locations and relative lateral, torsional, and longitudinal stiffnesses. Much research on predicting wheel load distributions is under way, but the scatter in analysis is always greater than what could be obtained for an existing bridge by direct measurements.

2. The impact allowance specified in AASHTO is usually higher than that obtained by testing. Moreover, the observed value is often a function of maintenance (bump or roughness) rather than span length as given in AASHTO.

3. Truck traffic and weight distribution will vary considerably from site to site. These parameters affect the likelihood of occurrence of extreme combinations of truck overloads. For example, the probability of occurrence of very heavy and closely spaced vehicles is affected by the truck weight distribution, volume, percentage of side-by-side occurrences, and inspection interval.

The most direct way to consider these observations in the rating process is by using field strain measurements to determine girder distribution and impact factors. The values that are presented by AASHTO are by nature conservative and intended to fit a variety of situations. For a specific bridge the field data will obviously give more exact values.

The approach proposed in this section is simply to modify the AASHTO design girder distribution and impact factors and substitute the field-measured values. In general, this approach will change the live load stresses by about 30 percent for the distributions and about 15 percent for the impact. (Examples will be given later.)

According to the maintenance manual, the safety factors should be increased if the site contains a large volume of vehicles or many heavily loaded trucks. These traffic-related variables are addressed in the subsequent section in which structural reliability techniques are used in assessing the safety of bridge members.

SAFETY ANALYSIS

Bridge safety compares loading demand on the component and strength capacity. Safety factors are needed to account for the possibility of overloads; inaccuracies in calculation of load effects; and variability of material properties, fabrication, and other tolerances. Modern safety theory has as a goal the quantification of uncertainties and the application of design factors that produce structures with uniformly consistent reliabilities. For new bridges, the safety factors reflect the long-term distributions of maximum load and deterioration of member strengths. For rating bridges, the safety factor can be lower because the exposure period is shorter, knowledge of loading is more precise, and member deterioration can be revealed by inspection and corrected.

The basic reliability model is used to examine the safety margin (4):

$$g = R - S \tag{1}$$

where

- g = safety margin;
- R = member strength capacity; and
- S = load effect on the member; S includes the dead (D) and live (truck) loads (L):

$$S = D + L \tag{2}$$

The component is safe as long as the safety margin (g) is positive. A convenient measure of safety is the reliability or safety index (β). This includes the mean value of g and its uncertainty as expressed by its standard deviation (σ_g):

$$\beta = \bar{g}/\sigma_g \tag{3}$$

The standard deviation of g for a simple case is obtained from the expression for a sum:

$$\sigma_g^2 = \sigma_R^2 + \sigma_D^2 + \sigma_L^2 \tag{4}$$

where σ_R, σ_D, and σ_L are the respective standard deviations of R, D, and L. In the bridge rating example, σ_R will depend on the variability of material properties as well as the estimation of scatter in material deterioration. σ_D depends on the dead weight including estimations of overlays; and σ_L depends on truck weight parameters, volume, girder distribution, and dynamic amplifications.

A model presented previously for calculating safety indices is briefly reviewed here (5). The maximum live load effect (L) at a position along a bridge is expressed as

$$L = a\ m\ W^*\ H\ g\ I \tag{5}$$

where

- a = constant based on span length and configuration of design vehicle;
- m = variable that reflects the randomness in the axle configurations of representative random truck traffic;
- W^* = variable that corresponds to the weight of the upper 5 percent of the gross weight histogram; this magnitude, which is typically in the 60- to 80-kip range, was found to adequately represent the severity of truck weights at a site;
- H = multiple presence or headway variable to reflect the ratio of maximum moment to a load caused by a design weight equal to W^*;
- L = live load effect including all loaded lanes; L includes the likelihood of side-by-side occurrences or multiple presence on the bridge and the extreme tail of the weight histogram;
- g = girder distribution factor; and
- I = impact factor.

The values of the means and standard deviations of all the random variables listed can be obtained from observations of the capacity of the bridge members and the traffic crossing the bridge as will be illustrated in a later section.

WEIGH-IN-MOTION SYSTEM

The data needed for the examples in this paper were obtained using the weigh-in-motion (WIM) system developed at Case Western Reserve University under the sponsorship of the Ohio Department of Transportation and the Federal Highway Administration (6,7). The system uses existing bridges as equivalent static scales to obtain unbiased truck gross and axle weights, classification, dimensions, and speed. For the purpose of this study, the WIM system provided the following information on the vehicle traffic on the instrumented bridge:

1. A total count of the vehicles that cross the bridge;
2. The lane traveled by each vehicle;

3. The time of arrival of each vehicle on the bridge;
4. The speed of each vehicle;
5. The truck type, found from the axle configuration of vehicles heavier than 10 kips;
6. The spacings between the axles of each truck;
7. The weights of the trucks' axles; and
8. The trucks' gross weights.

In a second step, information on the behavior of the bridge members is obtained by observing

1. The stresses at the gauge locations,
2. The girder distribution factors, and
3. The dynamic amplifications of the strain records.

This information is assembled in histograms and the means and standard deviations of the variables are calculated and used as illustrated in the examples given in the next sections.

RATING EXAMPLE

Five sites were instrumented to illustrate this study. All five bridges had parallel steel girders and gave fairly typical representation of bridges in Ohio in terms of design (both composite and noncomposite bridges were instrumented) and truck traffic composition (posted bridges and bridges with high permit loads were evaluated). In this section, the results for one site are presented; the results for the other sites are given elsewhere (8). This bridge on I-475 in Lucas County was chosen because of the large number of heavy special vehicles that crosses it. The layout of the six-girder 176-ft bridge is shown in Figure 1. The bridge was overdesigned according to AASHTO specifications and was allowed high levels of permit trucks.

Truck Traffic

More than 600 trucks were observed during the 4 hr of continuous data acquisition. Seventy-six percent of these trucks were in the right lane. The number of side-by-side occurrences is an important factor in determining the maximum expected load on a site. For the purposes of this study, all trucks that run over the bridge within 0.5 sec of each other in the two lanes are considered to constitute one side-by-side occurrence. It was observed that 0.7 percent of the trucks were in the left lane following, within 0.5 sec, a leading truck in the right lane. Also, 0.3 percent of the trucks were in the right lane within 0.5 sec of a leading truck in the left lane. This indicates that about 1 percent of all truck occurrences are side-by-side events. This value is consistent with measurements at other sites (8). The truck gross weight histogram for this site is shown in Figure 2. The mean of the loaded semitrailer trucks (heavier than 20 kips) is 45 kips. The 95th percentile weight (W*) is 78 kips, which indicates a relatively high loading distribution compared with other sites in Ohio. The 95th percentile weight excluding permit trucks is 74 kips.

Measured Stresses

Part of the WIM operation consists of a "calibration" phase during which the strain record of a "test" truck of known weight is recorded. The 29-kip truck used on this bridge produced a maximum stress of 1.12 ksi on the third girder when the truck traveled in the right lane. The stresses were measured on the first span at a location corresponding to the lowest rating value as determined by the standard AASHTO procedures. The stresses on each girder from several other truck crossings in different lateral positions are given in Table 1. Table 2 gives the measured stresses due to the heaviest trucks recorded at this site. It can be observed that the maximum single girder stress is 3.40 ksi, which was caused by a 136.7-kip truck with five axles.

Girder Distribution

Random traffic with a minimum gross weight of 20 kips was used to determine girder distributions for each lane (Table 3). The distribution factors for side-

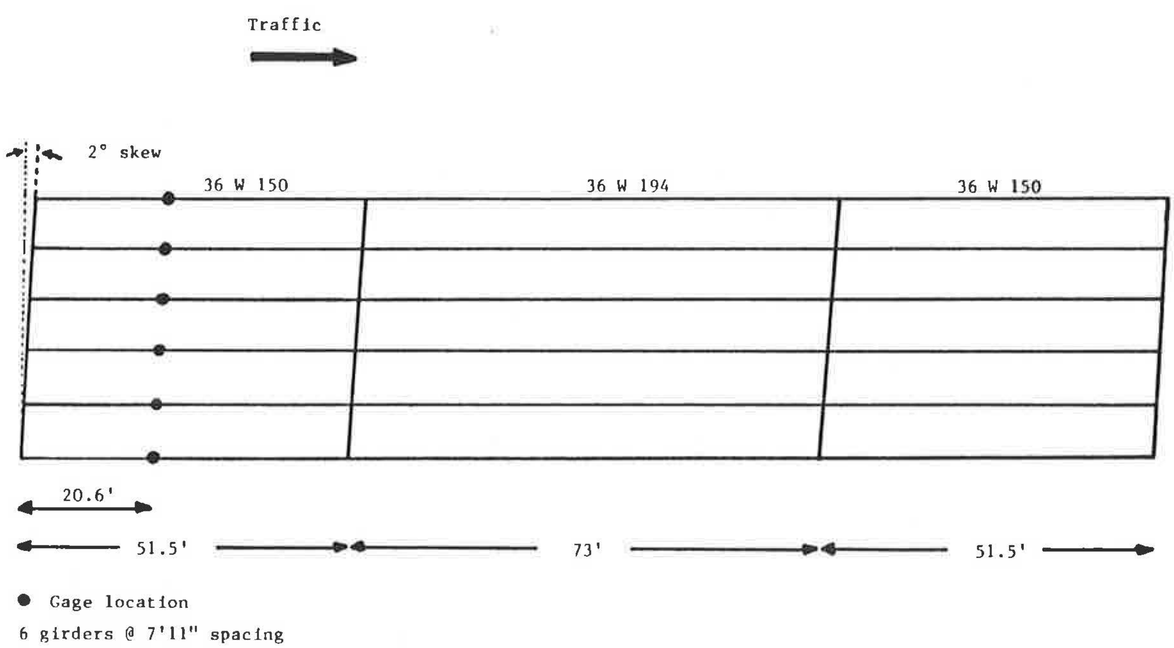

FIGURE 1 Layout of I-475 site.

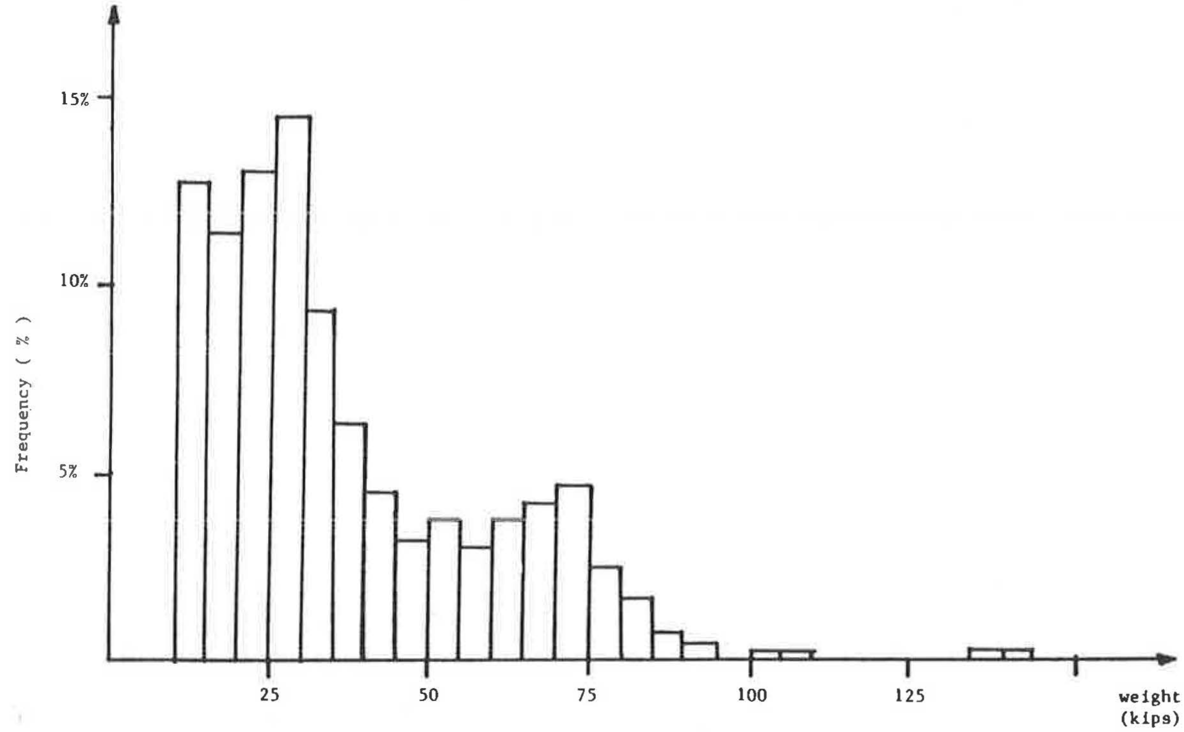

FIGURE 2 Truck weight histogram.

TABLE 1 Test Truck Stresses

Record	Location	Stress (ksi) at Girder 1	2	3	4	5	6
2	Right lane	0.30	0.78	1.10	0.69	0.54	0.13
11	Right lane	0.21	0.75	1.12	0.70	0.47	0.10
12	Right lane	0.22	0.71	1.03	0.68	0.49	0.21
13	Left lane	0.07	0.27	0.44	0.82	0.93	0.57
14	Left lane	0.02	0.20	0.37	0.75	0.93	0.57
15	Left lane	0.12	0.33	0.50	0.87	0.98	0.57

Note: Test truck weight = 29 kips.

by-side occurrences are found by summing the average distributions (plus one standard deviation) from each girder for both lanes. The most heavily loaded girder was determined to be the fourth girder with a distribution factor of 54 percent of a single lane load. The calculation is executed as follows:

21 percent (distribution of Lane 1) + 3 percent (standard deviation Lane 1) + 28 percent (distribution of Lane 2) + 2 percent (standard deviation Lane 2).

TABLE 2 Random Heavy Truck Stresses

Weight (kips	Moment (kip-ft)	Axles	Stresses (ksi) Measured	Composite[a]	Noncomposite[a]	Noncomposite[b]
136.7	707	5	3.40	12.14	7.70	3.32
141.4	568	11	2.17	9.75	6.19	2.66
141.9	623	11	2.39	10.69	6.78	2.92
138.3	513	11	2.40	8.82	5.59	2.41
106.1	496	6	2.17	8.51	5.40	2.32
103.8	562	6	2.07	9.65	6.12	2.63

[a]Using AASHTO distribution factor.
[b]Using average measured distribution factor (31 percent).

TABLE 3 Girder Distributions (%)

	Girder 1	2	3	4	5	6
Lane 1						
Average[a]	7	23	31	21	14	4
COV	3	3	4	3	3	3
Lane 2						
Average[a]	2	7	11	28	33	19
COV	2	2	3	2	3	3
Total	14	35	49	54[b]	53	29

Note: COV = coefficient of variation.
[a]Average of random vehicles with weights greater than 20 kips.
[b]Maximum distribution factor.

The 49 percent distribution and the 54 percent distribution used herein compare favorably with the applicable AASHTO value of 72 percent [7.92/(2 x 5.5)].

The standard deviation is added to the average distribution of each lane to account for possible situations in which the girder under consideration supports a higher than average percentage of the total load. The 54 percent distribution factor is an extrapolated value that would exist if two identical vehicles were exactly side by side.

Dynamic Impact

The dynamic responses for the test truck and heaviest vehicles were estimated from the response strain record. These are generally under 10 percent and are significantly less than the 28 percent prescribed by AASHTO (Figure 3). The one exception noted to the 10 percent value is a 106.2-kip vehicle with six axles that had a dynamic response of 15.4 percent. The dynamic response for the I-475 bridge was taken as 10 percent for the rating calculations.

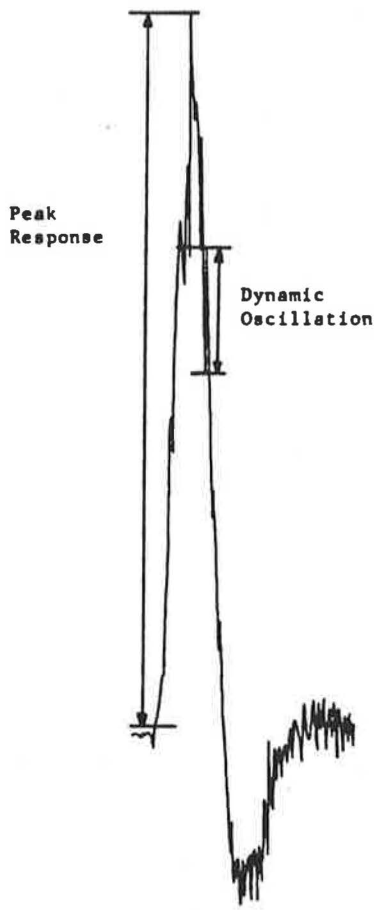

FIGURE 3 Example strain record.

Computed Stresses

The maximum measured stresses were compared with calculated stresses from the influence curve for selected heavy trucks (Table 2). The results confirm that the bridge provided a large composite action even though it was designed as a noncomposite bridge. For example, the 136.7-kip, five-axle truck produced a measured maximum single girder stress of 3.40 ksi. This value compares with a calculated stress of 12.14 ksi assuming noncomposite action and 7.70 ksi assuming composite action and using AASHTO's girder distribution factor. If the average measured girder distribution factor (31 percent) for Lane 1 is used, the girder stress assuming composite action is 3.32 ksi. Similar differences between maximum measured stresses and computed stresses have appeared in all the noncomposite design sites surveyed. It should be noted that, in all cases observed, the top flanges of the steel girders were partly encased in the deck, which provides shear transfer.

CONCLUSIONS--SITE 4

1. Measured stresses for the test truck and selected heavy trucks are lower than calculated stresses because of composite action, lower impact, and lower girder distribution factors than prescribed in AASHTO's specifications.

2. Using the measured values for impact and girder distribution increases the rating factors in comparison with those in ODOT's rating report. For example, using operating stresses of 27 ksi and looking at the middle point of the center span, a rating factor of 1.94 is calculated using field measurements. This value compares with 1.28 calculated using AASHTO's impact and girder factors. In both of these calculations noncomposite sections are assumed. Table 4 gives a comparison of the rating factors as given in ODOT's report and the rating factors obtained when field measurements are used.

TABLE 4 Comparison of Rating Factors for Five Sites

	Rating Factor	
Site	AASHTO	Field Measurements
1	1.42	1.75
2	1.59	1.50
3	1.57	2.06
4	1.28	1.94
5	2.34	3.46

Note: Noncomposite action assumed except for Site 5.

3. It should be noted that Site 4 had extremely heavy vehicle traffic compared with other sites observed in Ohio. Thus consideration should be given to raising the load factor to account for additional uncertainty in maximum loading. This factor and reliability-based procedures to account for load uncertainty are discussed in the next section.

SAFETY LEVEL FOR RATING

Measured data are available for calculating the uncertainties of the random variables discussed earlier. As an illustration of the procedure that can be used to evaluate the safety of an existing bridge (rating), the information collected at the Lucas I-475 bridge is used in the following example.

First, a deterioration factor is introduced as a random variable to reflect uncertainty about losses of girder section over the years. The deterioration factor (Det) expresses the percentage of the original section still capable of carrying load; a factor of 1.0 implies that no section loss was detected for the bridge's girders. This Det factor is also expressed with some uncertainty. Because the rating report for the I-475 structure did not indicate any deterioration in the steel members, a Det factor of 1.0 is used for the mean of this variable, and a standard deviation equal to 5 percent of the mean [coefficient of variation (COV) = 5 percent] is associated with Det to model the uncertainty in the evaluation procedure. The term R in Equation 1 is then replaced by Det x R where R is the original resistance of a member as provided by the plans. The nominal value of R as calculated from the site plans for a girder in the middle span is equal to 1,990.5 kip-ft. Coupon tests on rolled beam members, however, show that the average stress capacity of A36 steel is closer to 40 ksi not 36 ksi, which suggests that the mean member capacity of a girder at the midspan of the bridge is really 2,212 kip-ft, the value used in this example. A COV of 8 percent is associated with R to reflect uncertainty in the steel yield stress, section dimension, and so forth.

The dead load (D) in Equation 2 was estimated by the bridge engineers to be 316.9 kip-ft. A COV of 5 percent is herein associated with D to reflect the level of confidence of this estimate. In reality, this COV should be based on the level of effort made to estimate the existing dead loads acting within the structure.

The I-475 site is a "special" route for extremely heavy permit vehicles; three different cases of maximum expected live loads are considered:

1. Random vehicle occurrences,
2. Combinations of random vehicles and permit trucks, and
3. Combinations of permit trucks only.

Random Vehicle Occurrences

The expression of Equation 5 is used to predict the maximum expected load due to random vehicles:

"a" is a factor that gives the moment effect of the design vehicle at the point being analyzed. For the midspan, the semitrailer vehicle or Rating Vehicle b (Figure 4) controls the design and is used to calculate "a." This "a" factor is known precisely and has no uncertainties associated with it. "a" is calculated for a truck of one unit load and thus is a reflection of the axle spacings and axle weight distributions of Truck b rather than of its total weight.

"m" represents the variation between the effect of random trucks and the effect of the rating vehicles specified in the analysis. From the truck data collected at this site, it was found that "m" at the midpoint of the middle span has a mean of 0.94 and a COV of 14 percent. This means that, on the average, the rating vehicle overestimates the effect of the random vehicles that crossed the bridge by 6 percent.

W* is a characteristic weight calculated from the gross weight histogram of the semitrailers collected at this site. W* is calculated so that it gives the upper weight limit of 95 percent of the trucks counted. A value of 74 kips was measured when the permit loads were excluded and it is associated with a COV of 5 percent. The latter uncertainty is based on comparison with similar sites and also reflects the limited test period (4 hr of data).

H reflects the effect of multiple occurrences and is calculated using simulation techniques (8). The H-value calculated for this site is 3.14 with a modeling uncertainty or COV of 10 percent. This value of H corresponds to an expected maximum occurrence of 232 kips (3.14 x 74) on the bridge at one time during the 2-year period. The 10 percent uncertainty implies that there is a 5 percent chance that this weight might actually exceed 279 kips.

For the girder distribution factor (g) for this site, the fourth girder counting from the extreme girder of the main lane is the most critical member. From the field measurements, it was found that, on the average, this member carries 24 percent of the total load (49 percent of one lane load) on the bridge with a COV of 7.5 percent.

From the strain traces of random heavy vehicles it was found that the maximum total effect (static + dynamic) of the impact factor (I) for this site is on the average 1.06 times the static effect with a COV of 8 percent.

The means and standard deviations of the random variables used in this example are given in Table 5. The safety index (Equation 3) calculated for this site under random vehicle crossings is 6.96, which indicates an extremely high safety level. The failure function used in these calculations is

$$g = \text{Det } R - D - a\ m\ W^*\ H\ g\ I \qquad (6)$$

Typically, new design codes such as those of Ontario (9) or the American Institute of Steel Construction are calibrated to achieve lifetime safety indices on the order of from 3.0 to 3.5. The high safety factor calculated in this example is not surprising considering that the bridge has a rating

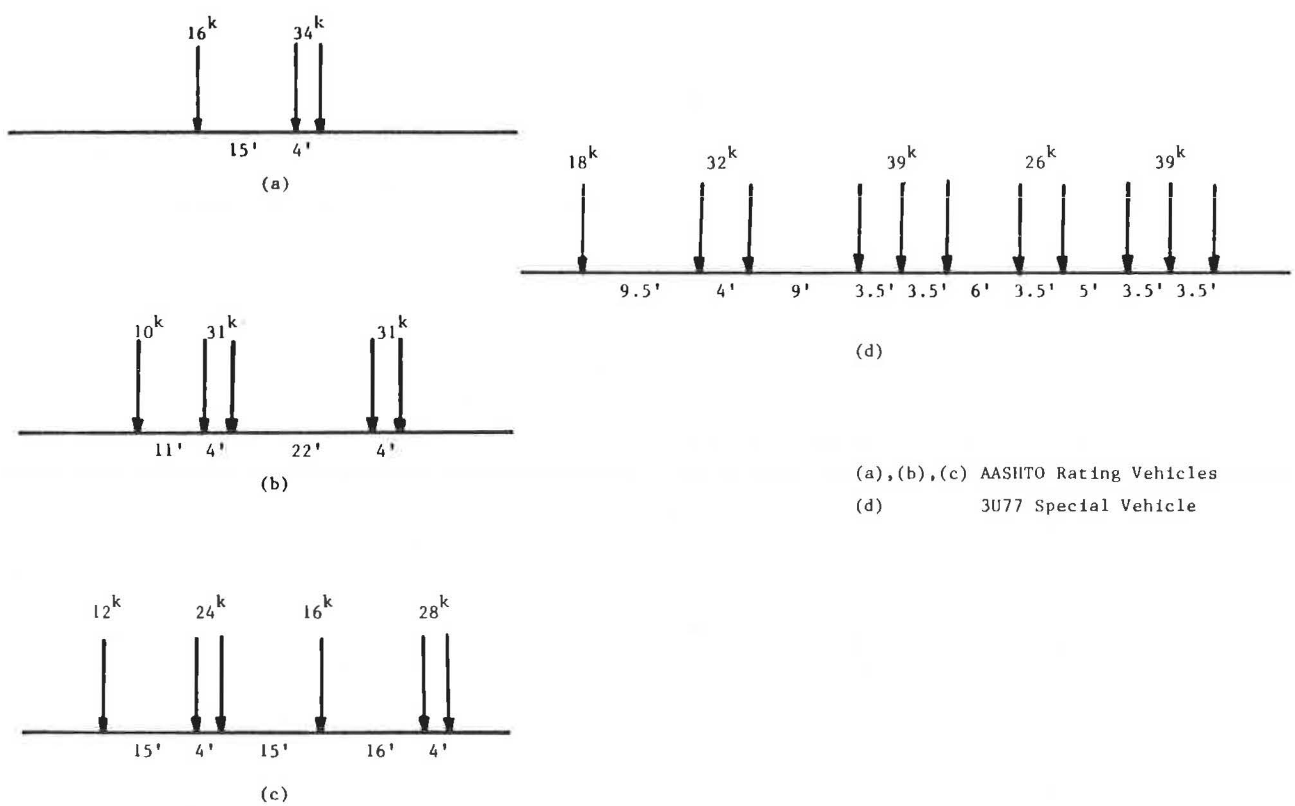

FIGURE 4 Examples of special vehicle configurations.

TABLE 5 Summary of Input Data for the Safety Evaluation

Variable	Mean	COV (%)
R	2,210 kip-ft	8
Det	1.0	5
D	316.9 kip-ft	5
W*	74 kips	5
H	3.14	10
m	0.94	14
g	0.24	7.5
I	1.06	8
α	1.21	10
C_h	1.03	5
P	1,020 kips	5

factor of 1.28 based on AASHTO's working stress design method.

Combinations of Random Vehicles and Permit Trucks

Because of the large number of permit vehicles at this site, the possibility of a permit vehicle alongside a heavy random truck is likely to control safety. The method by which this possible combination is considered in the safety analysis is detailed elsewhere (8). Det, R, D, W*, g, I, m, and a factors are calculated as previously described. Because in this situation one of the side-by-side vehicles is known, H is replaced by an α factor that describes the overload due to a random vehicle in only one lane.

For this site, assuming 10,000 permit vehicles in 2 years, α was found to be equal to 1.21 and is associated with a COV of 10 percent to reflect uncertainties in the modeling of the live load and limitations in available site data. The failure equation or safety margin (g) now takes the following form:

$$g = \text{Det } R - D - (a\, m\, W^*\, \alpha + P)\, C_h\, g\, I \quad (7)$$

where

- α = lane overload factor,
- P = weight effect of a permit vehicle, and
- C_h = headway correction factor for vehicles ahead of and behind the permit truck in the random vehicle combination.

The other factors are as defined earlier. The values of the means and standard deviations used in the safety index calculations are given in Table 5.

P at this site is due to the effect at the middle point of the midspan of a 150-kip permit vehicle with the axle configuration shown in Figure 4. It is associated with a COV of 5 percent to model the possibility that some of the permit vehicles may have slightly different axle weight distributions or axle configurations. A C_h of 1.03 with a COV of 5 percent is obtained from Moses and Ghosn (5). The safety index calculated using the information for this site is 8.42. This is even higher than the 6.96 calculated for random vehicle occurrences because some of the random vehicles showed gross weights as high as the permit weight and axle configurations that produced higher moment effects.

Combinations of Permit Trucks

Because of the large number of permit vehicles at this site, this possible combination should be considered in the safety evaluation of this bridge. The method used here first calculates the safety index of a bridge member, given the occurrence of this combination, then modifies this safety index to include the probability of such an occurrence. The latter probability depends on the number of permits issued, which is assumed to be 10,000 a year for this example (8). The safety index for the fourth member at the middle point of the middle span of this bridge is calculated using the safety margin equation:

$$g = \text{Det } R - D - 2\, P\, C_h\, g\, I \quad (8)$$

Equation 8 is similar to previous safety margin models (Equations 6 and 7) with the exception that the live load is due to the effect of two permit loads represented here by the term 2 P. A safety index of 7.26 is calculated for this condition. The safety index is conditional on an actual occurrence of two permit loads side by side. Given that 10,000 permit crossings are expected in 2 years, with 1 percent of these crossings occurring side by side, and that 2.9 million crossings (4,000 per day) of random trucks are expected in the 2-year rating period, the probability of having two permit vehicles side by side is calculated to be 30 percent. Combining this probability with the conditional safety index of 7.26 yields a final safety index of 7.42.

Combining the safety indices for the random-random, random-permit, and permit-permit combinations yields an overall safety index of 6.95. This safety index, which is much higher than the acceptable range of from 3.0 to 3.5, indicates high safety levels. This is due to several circumstances including the overdesign of the bridge by some 30 percent at some locations and the apparent good maintenance (there was no member deterioration detected). The well-kept pavement preceeding the bridge produced low dynamic impacts on the bridge members. All of these factors combined increase the level of confidence that a bridge rating engineer should have in the safety of this bridge. It should be noted, however, that no fatigue evaluation is being undertaken here and that consideration of fatigue failures might decrease the safety level.

CONCLUSION

The field data demonstrated the following conclusions for the five sites surveyed:

1. The maximum stresses were significantly below values predicted by conventional procedures. The major reasons for this observation are (a) all sites behaved with composite action, though only one had been constructed in this manner; (b) additional contributions to section modulus may result from overlays, parapets, curbs, steel in the concrete deck, and the like, which are not normally considered in analysis; (c) girder distributions are more conservative than predicted by AASHTO values; only in the case of closely spaced girders (5.5 ft) were the AASHTO values exceeded; (d) measured impact values are lower than AASHTO's, presumably because the sites had relatively smooth surfaces although this was not a factor in site selection; and (e) the occurrence of extremely heavy trucks in both lanes simultaneously is rare.
2. The rating factor computed after incorporating the measured data usually exceeded the values obtained by conventional rating procedures.
3. Numerous cases in which truck weights exceeded legal loads were observed; however, these did not impair overall bridge safety.
4. Predicted reliability levels for the sites studied were high, though only strength, not fatigue, was modeled.

5. All the bridges studied were redundant multigirder systems; for this type of bridge a safety index of from 3.0 to 3.5 provides an acceptable level. For nonredundant bridges, the target safety index should be much higher.

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REFERENCES

1. Better Targeting of Federal Funds Needed to Eliminate Unsafe Bridges. GAO-CED-81-126. Government Accounting Office, Aug. 1981.
2. Manual for Maintenance Inspection of Bridges. AASHTO, Washington, D.C., 1982.
3. Standard Specifications for Highway Bridges. AASHTO, Washington, D.C., 1983.
4. P. Thoft-Christensen and M.J. Baker. Structural Reliability Theory and Its Application. Springer-Verlag, New York, 1982.
5. F. Moses and M. Ghosn. A Comprehensive Study of Bridge Loads and Reliability. Final Report FHWA/OH-85/005. FHWA, U.S. Department of Transportation, Jan. 1985.
6. F. Moses and M. Ghosn. Weighing-Trucks-In-Motion Using Instrumented Highway Bridges. Final Report FHWA/OH-81/008. FHWA, U.S. Department of Transportation, Dec. 1981.
7. F. Moses and M. Ghosn. Instrumentation for Weighing-Trucks-In-Motion for Highway Bridge Loads. Final Report FHWA/OH-83/001. FHWA, U.S. Department of Transportation, Aug. 1983.
8. F. Moses, M. Ghosn, and J. Gobieski. Weigh-In-Motion Applied to Bridge Evaluation. Final Report FHWA/OH-85/012. FHWA, U.S. Department of Transportation, Sept. 1985.
9. Ontario Highway Bridge Design Code. Ontario Ministry of Transportation and Communications, Downsview, Ontario, Canada, 1979.

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