Structural Reliability of Bridges Designed Using HS25 in the State of Michigan

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Abstract

This paper presents the process and results of a research project to examine the adequacy of current design loads for bridges in the state of Michigan. The concept of structural reliability is used here for this evaluation. The target reliability index of 3.5 for calibrating the AASHTO LRFD Bridge Design Specifications was used in this study as the criterion for evaluating the adequacy. Reliability indices were calculated for twenty structures randomly selected from the Michigan inventory of new bridges. This bridge suite included five bridges from each of four major types in Michigan: steel girder, prestressed I-beam, prestressed adjacent box girder, and prestressed spread box girder bridges. Weigh-in-motion data were processed to statistically characterize truck load effect in beams for moment and shear at critical sections. The reliability analysis was also kept separated by the functional class of roadway. For moment and shear in beams or girders, two strengths were used in the reliability analyses: 1) strength as designed according to construction plans termed herein as as-designed; and 2) strength required by the current Michigan design code termed herein as design-minimum. The two different girder resistances resulted in different reliability levels, offering some insight into the amount of over-design typically practiced in Michigan. The reliability indices were found to vary significantly among the bridge types and the as-designed reliability indices were found, on average, to meet the adequacy criterion. However, many design-minimum reliability indices did not meet the criterion for several functional classes of roadway.

INTRODUCTION

The load carrying capacity of bridges is clearly influenced by the design load used in their design. The design load also has a significant effect on the durability of these bridges. Traditionally, the design load adopted in the design specifications is applied uniformly within the jurisdiction of a transportation agency, with some exception. For example, a state typically uses one design load level for most bridge designs in the state, with a possible exception for bridges that have a particular function or characteristic that may warrant a different design load, such as those on certain local roads. This practice has been justified in that it reduces required engineering design work and avoids bridge-specific design-load. On the other hand, this approach also neglects location-specific truck loads that may be substantially different from bridge to bridge. This issue becomes critical when actual truck loads are noticeably higher than the design load. The motivation for this study was that bridges experiencing these higher loads are subjected to a higher risk of distress, damage, and even failure.

Background

In 1972 the Michigan Department of Transportation (MDOT) changed the design load level for all bridges located on the Interstate and Arterial Highways from HS20 to HS25. Currently MDOT still uses the HS25 load for bridge beam design. The legal truck load in Michigan is higher than many other states, while the legal axle load is consistent with other states. The objective of this study was to address a concern as to whether the actual truck loads are adequately accounted for by the current bridge design load in the state of Michigan. This was done because Michigan has significant diversity of industries in different areas, resulting in significantly different truck load spectra throughout the state. The concept of bridge structural reliability was used in this study to perform the evaluation of current design load for bridges in Michigan.

Structural reliability is measured in this study using the structural reliability index β, which has been used in several recent research projects related to bridge safety (1,5,8), including NCHRP Project 12-33 Development of LRFD Bridge Design Specifications. The structural reliability index is a measure of the safety reserve in a structural system. In that project (1), the LRFD bridge design code was calibrated using the structural reliability index β. Note that a large β indicates a higher reliability level. For evaluation of the design load, this research effort covers only bridge superstructures. The design load is determined using the load factor design (LFD) method contained in the AASHTO standard specifications (2). This is the design standard currently used for bridge design in Michigan.

In this study, a survey of the Michigan bridge inventory found that the following four superstructure types represent 91% of the new bridges built in the past 10 years. 1) Steel beam bridges (40.0%). 2) Prestessed concrete I beam bridges (30.6%). 3) Adjacent prestressed concrete box girder bridges (14.6%). 4) Spread prestressed box girder bridges (5.6%). Accordingly, these four superstructure types were included in the present study, representing the current and foreseeable future population of new bridges in the state. Each of these bridge types has a configuration consisting of a concrete deck supported by beams. For each of these four types, 5 bridges were randomly selected from those built in the past 10 years. This sample of 20 bridges was then used in this study to represent the new bridge population, particularly to provide information on dead load effects, “as-designed” capacities, span lengths, etc., for the reliability analysis. The analysis was performed for interior beams of the selected bridges, as well as for the reinforced concrete decks. This paper focuses only on the reliability analyses and results for the beam shear and moment effects, due to the limit of space.

It is well known that the strength of a specific bridge’s component is allowed to be higher than what is required by the design specifications. Depending on a number of factors, this additional amount of strength may be substantial. Some of the influencing factors for this situation are as follows. 1) The designer may consciously exercise conservatism in design, leading to a higher strength than required. 2) A particular load effect may represent a non-dominant failure mode. Thus, the strength for that load effect can become highly excessive. For example, shear may be a non-governing load effect for bridge beams having a long span. As a result, the shear capacity provided by a cross section can be much higher than required if the section already meets the moment requirement. Accordingly, in this research project, the reliability index β is computed for two cases of strength. Specifically, the one required by the current design specifications is termed as “design-minimum”, and the other as designed by the designer is termed as “as-designed”. Comparison of these two β values for the same component can show the influence of reserve strength on structural reliability, possibly provided in current design practice but not required by the design specifications.

Concept of Structural Reliability

In this study, the structural reliability of a bridge component is evaluated using its failure probability defined as:

$$\text{Failure Probability} = P_f = \text{Probability [Resistance – Load Effect} < 0 \text{]}$$

$$= \text{Probability [} R - S < 0 \text{]} \qquad (1)$$

where resistance R is the load carrying capacity of the structural component, and S is the load effect or load demand on the component. For example, the load effect can be bending moment for a beam section and the resistance is then the section's moment capacity. The resistance and load effect in Equation 1 are modeled as random variables because they both possess some level of uncertainty. In general, the uncertainties associated with the resistance are due to variation in material properties, quality of preparation process, construction quality, etc. The uncertainty associated with load effect is related to randomness in truck weight, truck type, traffic volume, etc. Note that the failure probability in Equation 1 refers to a load effect in a structural component. Namely, this definition can be applied to a variety of load effects, such as moment, shear, or even possibly displacement if serviceability is the focused issue. The equation also can be applied to a variety of bridge structural components, such as beams, slabs, piers, etc.

The so-called reliability index β can be expressed in terms of the failure probability given in equation 1 as

$$\beta = \Phi^{-1}\left(1 - P_f\right) \tag{2}$$

where Φ^{-1} is the inverse of the standard normal random variable's cumulative distribution function. Equation 2 indicates that β is inversely monotonic with P_f. Namely, a small P_f leads to a large β, or a large P_f to a small β. Thus, a large β indicates a more reliable structural component and a small β a less reliable one. More details of computing β can be found in (7,8).

The AASHTO LRFD Bridge Design Specifications (3) was calibrated using the same concept of bridge structural reliability (1). That research effort used the same method of calculating structural reliability index β as that used here. The target reliability index used in that study was 3.5. Note that this selection of target reliability index is somewhat arbitrary. In that study, the value of 3.5 was selected to provide the same average safety margin in the LRFD code that was estimated to exist in the AASHTO standard bridge design code. Thus, it is appropriate to state that the particular target value 3.5 reflects an average of reliability levels typically practiced in the country over several decades.

The present research project used the same structural reliability index concept to assess bridge structural reliability assured by the current Michigan design load subjected to current operating loads of trucks. In addition, all statistical parameters including the mean and standard deviation of the involved random variables were kept as consistent as possible with (1) except the live load effects, which were estimated from Michigan-specific data. The target β value of 3.5 may be used as the criterion for evaluating the design load. The truck loads used in the reliability analyses in this study were modeled based on weigh-in-motion (WIM) truck weight data gathered in Michigan (4).

STATISTICS FOR LIVE LOAD EFFECT, DEAD LOAD EFFECT, AND RESISTANCE

Live Load Effect Statistics

Weigh-in-motion (WIM) truck weight data (4) were retrieved for this study for five different functional classes (FC) of roadway. They included 1) FC01: Principal Arterial – Interstate Rural; 2) FC02: Principal Arterial – Other Rural; 3) FC11: Principal Arterial – Interstate Urban; 4) FC12: Principal Arterial – Urban; and 5) FC14: Other Principal Arterial – Urban. A total of over 46,000 trucks were included in these WIM datasets, although this number was slightly reduced to maintain consistency with the data preprocessing practice of (1). Accordingly, trucks meeting either of the following two criteria were eliminated from the datasets for further analysis: 1) having two axles weighing less than 10 kips; and 2) having three or more axles weighing less than 15 kips. In reliability analysis this allows the larger loads to be modeled more accurately with a statistical distribution. The resulting live load data were found to be well modeled by a lognormal distribution.

It is known that load effect in a bridge component depends on the arrangement of the component in the bridge's structural system. To realistically model this, five bridges of each of these four types were used to extract the required information: steel beams (SC), pre-stressed concrete I-beams (PI), pre-stressed concrete spread box girders (PCS), and pre-stressed concrete adjacent box girders (PCA). All of these 20 bridges are designed with a composite concrete deck. The bridges were selected randomly from the Michigan bridge inventory but were limited to those constructed or re-constructed within the past 10 years, as discussed earlier. Recall that the purpose of this requirement was to ensure that the structural reliability estimates would be representative of the population of new bridges. Some details of these bridges, including their locations within Michigan and the number of spans, are shown in Table 1.

For modeling flexure and shear effect for the truck load, moment and shear influence lines were developed first for each bridge's critical sections. An influence line for each section and each load effect were combined to obtain live load effect data for that beam section and load effect. Then every truck in the WIM dataset was "run" through the influence line to find the truck's maximum load effect, using a computer program. The results of maximum load effect for all the trucks consequently provided a set of data for modeling the random variable of that load effect. For example, when a two-span continuous steel composite (SC) bridge was modeled, the critical sections for moment were identified as the 0.4 points from the two respective end supports for positive moment, and the center support for negative moment. Those for shear are the three support sections. Then the influence lines for these critical sections were generated. For each influence line, after all the trucks in the WIM dataset had been used in this simulation process, a set of maximum live load effects was obtained to generate the statistics for that load effect. This was also done separately for each functional class (FC) of roadway. The main reason for keeping the data and subsequently the reliability analyses separated by FC is that the volume of truck traffic on a roadway may be influential in projecting the live load effect to cover an expected life span (75 years) of new bridges. This traffic volume is quantified using average daily truck traffic (ADTT).

Once the live load effect statistics for each critical section on each bridge was determined as described the data was projected to form a 75-year statistical distribution. The 75-year statistical distribution for the load was needed for the reliability analysis. This projection depended on the total traffic volume over the 75 year period, which in turn is related to ADTT for the particular FC. ADTT data was procured through MDOT's planning division according to FC. The statistics of ADTT for a large number of sites were determined for the Metro Region (Region 7) of the state, within or near which the available WIM data were collected. This was done for FC01, FC11, FC12, and FC14, whose definitions have been given earlier. FC02 was neglected here because the Metro Region contains only a negligibly small percentage of roadways designated as FC02. Also note that the Metro Region includes the metropolitan Detroit area where a large population of industry and residents are located. In addition, the same ADTT data processing was done for the entire state (for all 5 functional classes including FC02) although no WIM data were available for areas other than the Metro Region and its vicinity. Using these ADTT statistics, the procedure described below was used to project the load effects of moment and shear for a 75-year period.

For a typical case, an "equivalent days of data" (EDD) was defined as

$$EDD = \frac{m}{ADTT} \tag{3}$$

where the numerator m is the number of trucks recorded in the WIM dataset for the interested FC, and the denominator is the ADTT. Since ADTT varies widely depending on which measurement site's data are used, it was decided to use two ADTT values (the 50th and 90th percentiles) to see the influence of ADTT variation on the reliability estimates. In addition, the Metro Region and the entire state were separately analyzed with the *EDD* values calculated. Thus, a total of four different groups of *EDD* levels were used in the reliability analyses for each FC.

The 50th percentile was chosen because it is the median, defined as the data point at which one-half of the data is below and the other half is above. The 90th percentile is a point where nine-tenths of the data are below and thus one-tenth is above. This percentile was chosen in order to observe a relatively extreme situation of high traffic and its effect on the structural reliability. This point also served as an indicator for the sensitivity of this parameter in the projection procedure and the overall reliability analysis. The selection of the 90th percentile was somewhat arbitrary, but was felt to be able to capture the extreme traffic occurrences for most sections of roadway in the state.

The projection of load effect to the expected life of 75 years was accomplished by projecting the cumulative distribution function (CDF) of the load effect obtained from the computer simulation discussed above. Typically for an FC and a critical section of a bridge, that simulation had resulted in a CDF based on the available WIM data. Using a binning scheme, this CDF can be expressed as follows

$$Probability\ [load\ effect < E_i]_{\text{for EDD days}} = F_i \qquad (i = 1, 2, \ldots, J) \tag{4}$$

where E_i is the midpoint of bin i of the load effect, being moment or shear, and J is the total number of bins. The empirical CDF (i.e., the F_i values) were found by sorting the truck load effect values (i.e., moment or shear) from smallest to largest. The corresponding value of the CDF for the i^{th} ranked bin of load effect can be expressed as

$$F_i = \frac{k_i}{m} \qquad (i = 1, 2, \ldots, J) \tag{5}$$

where m is the length of (i.e., the number of trucks in) the dataset and k_i is the number of trucks inducing a maximum load effect less than E_i, which approximately represents bin i. Details on an empirical CDF can be found in most statistics books.

For a single case of FC and a WIM dataset, the resulting CDF defined in Equations 4 and 5 is defined for an equivalent number of days as EDD in Equation 3. The projected CDF for 75 years, hereafter called $F_{i,75}$, was then calculated using the assumption that each time period of duration EDD within the life time of 75 years are statistically independent from one another. Hence, the projected CDF could be calculated as

$$Probability\ [\ load\ effect < E_i]_{\text{for 75 years}} = F_{i,75} = F_i^N \tag{6}$$

$$N = 75\ (365)\ /\ EDD = 27{,}375\ /\ EDD \tag{7}$$

where N is an exponent representing the number of time periods each having EDD days. Figure 1 shows two figures. The figure in the left window is a histogram for the bending moment that resulted from the WIM data for one of the bridges in the study for a single FC. By using the procedure described above, specifically equation 5, the solid line in the right window of figure 1 presents the empirical cumulative distribution function for the moment. Then, by applying equation 6 and 7, the cumulative distribution function projected to 75 years can be calculated and is presented as the dashed line in figure 1. Notice the straight portion of the dashed line, indicative of only several data points far out in the tail of the distribution.

Once the projected CDF was obtained for a particular load effect and a particular section, its mean and the standard deviation of the projected CDF were estimated. It should be noted that during the projection procedure there were several WIM datasets that did not contain a large number of heavier trucks that would have resulted in larger load effects. This led to a very small standard deviation for the projected 75-year CDF. It essentially indicates that within a 75-year period one may be relatively certain as to the largest load effect that may ever occur. Of course, an implicit assumption used here is that current truck weight limits will remain unchanged and thus the truck weight behavior will not change over time.

For the total live load effect on a bridge structural component, a multiplicative model was used to describe its' statistical characteristics:

$$L = T\ G\ I \tag{8}$$

where L is the total live load effect (moment or shear) as a random variable, T is the static load effect from the truck traffic obtained from the influence line analysis described above, G is the girder distribution factor for load sharing among parallel longitudinal beams, and I is impact factor to account for dynamic effect of moving load. Each of the quantities in equation (8) were modeled as random variables. The mean and standard deviation of T were obtained from the computer simulation process just described using WIM truck weight data. According to (1) and (5) the coefficients of variation (i.e. the ratio of the standard deviation to mean) V for I and G were taken as $V_I = 0.10$ and $V_G = 0.13$, respectively. Using an assumption of independent lognormal distributions for these random variables (5), the COV for the total live load effect, V_L, was estimated as

$$V_L = \sqrt{V_G^2 + V_I^2 + V_T^2} \tag{9}$$

where V_T indicates the COV of the static truck load effect calculated from the 75-year CDF presented above. Then the standard deviation σ_L of the total live load was readily calculated as the product of the mean of L and V_L. The total live load's bias defined as the ratio of the mean and nominal value was set at 0.9 according to (1). For the present study, an impact factor of I_n = 1.3 was used, where I_n is the nominal value of I. It has been found that this impact is mainly due to the roughness of the deck (5). It was thus reasoned that during the 75-year design life of a bridge the roadway surface would be significantly rough for some time, hence this value was selected.

A single lane loading girder distribution factor G was used in this project for modeling the live load effect. For example, it's nominal value is s/14 for the cases of steel and prestressed concrete beams (2), where s is the spacing between beams in feet. This selection was based on a previous research project of MDOT that found that the single lane girder distribution factor in the design code (2) is more realistic for modeling effects of truck load.

Dead Load Effect Statistics

The dead load effects were estimated from the plans provided by the MDOT for the 20 sample bridges. It was assumed to act as a uniformly distributed load. The critical beam was selected to be the one adjacent to the fascia beam, i.e. the first interior beam. Any loads resulting from safety railings or safety barriers located on the bridge edge were assumed to be distributed to the critical beam with a one-third factor. This was to account for load sharing among the beams. A 25 psf future wearing surface was included in the dead load effect for both the as-designed and design-minimum cases discussed earlier.

Each dead load effect was described statistically using an associated bias and coefficient of variation (COV). The dead load bias, D_{bias}, was expressed in terms of the nominal dead load, D_{nom}, and mean dead load, D_{mean}, as

$$D_{bias} = \frac{D_{mean}}{D_{nom}} \tag{10}$$

The bias and COV for the dead load effect was taken as 1.0 and 0.1, respectively (1). Since the nominal value of a dead load effect was estimated according to the bridge's plans, the mean value of the dead load effect was readily obtained by multiplying the nominal value by the bias, according to Equation 10.

Resistance Statistics

As mentioned earlier, beam capacities for moment and shear were determined for two different cases in this study. The first case was the as-designed case, in which the bridge plans used for construction were reviewed, and moment or shear capacities were computed. The second case was the design-minimum case, in which the minimum required capacities were computed based on the AASHTO bridge code (2) requirements including combined factored dead and live load (HS25) effects. For both cases, the resulting resistances were used as the nominal values in modeling the random variables' variation.

As-designed capacities for moment and shear. Basic principles of engineering structural analysis/structural mechanics were used to compute these capacities. These values were taken as nominal resistance for probabilistic modeling. It should be noted that no resistance factors (i.e., strength reduction factors) of any sort were applied to the calculated capacities. In order to maintain consistency with (1), ultimate strengths were determined based on composite sections without regard to construction staging.

Design-Minimum Capacities for moment and shear. The design minimum capacity for moment and shear was computed using the following procedure. The HS25 design load consists of a truck load with the axle weights of 10, 40, and 40 k (583 kN/m), or a lane load of 0.8 k/ft (11.67 kN/m) plus one or more point loads of 22.5 k (328.3 kN/m) (for moment) or 32.5 k (474 kN/m) (for shear), whichever results in a larger load effect. For example, in order to determine the *design-minimum* moment capacity for a simply supported bridge girder, one would compute

1) the maximum moment from the HS25 truck axle weights; and 2) the maximum moment from the combination of a 0.8 k/ft (11.67 kN/m) uniformly distributed load over the span plus a 22.5 k (328.3 kN/m) point load at the center of the span. Whichever is larger would be used as the nominal live load moment in the analysis.

The computation of design-minimum strength also required inclusion of the load distribution factor. For steel and prestressed I beam superstructure, the nominal live load moment, L_n, was then computed as

$$L_n = M_{L_n} \frac{s}{11} I_n \tag{11}$$

where M_{Ln} is the nominal static moment due to the HS25 loading, s is the beam spacing in feet. The denominator, 11, is the AASHTO distribution factor for steel and prestressed concrete I-beams for moment. The value I_n in equation 11 is the nominal impact factor to account for dynamic amplification of load effect due to a moving vehicle. The nominal moment capacity was then calculated based on the AASHTO load combination for LFD:

$$M_n = 1.3 D_n + 2.17 L_n \tag{12}$$

where D_n and L_n are nominal dead and live load effects and M_n represents the nominal strength required by the design code, or the design-minimum strength.

The bias and COV for the steel composite bridge girders was set at 1.12 and 0.1, respectively. The bias and COV for all other three types of prestressed bridge girders were taken as 1.05 and 0.075, respectively (1). The mean resistance was then calculated as the product of the bias and the nominal value. For the case of design-minimum strength, the nominal resistance was estimated using Equations 11 and 12. For the case of as-designed strength, it was computed using the information in the plans.

RELIABILITY INDEX CALCULATIONS AND RESULTS

Calculations

For the analysis of structural reliability using equation 1, the dead load and live effects were combined as $S = D + L$, the total load effect. The mean and COV of the total load effect S were then derived from the mean and COV of the dead load effect D and live load effect L. The following equation was used for this purpose, assuming that D and L are statistically independent of each other

$$\sigma_S^2 = \sigma_D^2 + \sigma_L^2 \tag{13}$$

where σ_D is the standard deviation of D determined as the product of the mean and the COV of dead load effect. Similarly, σ_L is the standard deviation of L determined as the product of the mean and the COV of live load effect. The mean value for the total load effect, S , is then the sum of the means of D and L as follows

$$\mu_S = \mu_D + \mu_L \tag{14}$$

where μ indicates the mean. The subscripts S, D, and L are respectively for total load effect, dead load effect, and live load effect as defined earlier. The COV of the total load effect can then be expressed as

$$V_S = \frac{\sigma_S}{\mu_S} \tag{15}$$

The reliability index β, defined in Equation 2, was calculated based on the statistics of the load effect S and the resistance R as

$$\beta = \frac{\ln(\mu_R) - \ln(\mu_S)}{\sqrt{V_R^2 + V_S^2}} \tag{16}$$

where μ_R and μ_S represent the means of the resistance and load effect, respectively. The values V_R and V_S represent the coefficient of variation of the resistance and load effect, respectively.

Details on the reliability analyses for each bridge can be found in (6). The reliability index β was calculated for the following eight different cases and the results are discussed in the next section.

1.) The as-designed girder capacity for the 50th percentile ADTT for the entire state.
2.) The as-designed girder capacity for the 90th percentile ADTT for the entire state.
3.) The design-minimum girder capacity for the 50th percentile ADTT for the entire state.
4.) The design-minimum girder capacity for the 90th percentile ADTT for the entire state.
5.) The as-designed girder capacity for the 50th percentile ADTT for Metro Region.
6.) The as-designed girder capacity for the 90th percentile ADTT for Metro Region.
7.) The design-minimum girder capacity for the 50th percentile ADTT for Metro Region.
8.) The design-minimum girder capacity for the 90th percentile ADTT for Metro Region.

Results

A total of about 3,000 β values were computed as previously outlined, for the listed cases of ADTT value, bridge location (in the entire state or the Metro Region), bridge ID, and cross section of the bridge. These β value ranged from as low as 1.2 to as high as 8.4 (6). The following approach was taken to synthesize the results and draw conclusions.

The controlling reliability index for a load effect (shear or moment) was determined for each bridge based on the-weakest-link-in-a-chain logic. For example, if *n* values of β were calculated for *n* cross sections in a bridge for moment, then the controlling reliability index for that bridge with respect to moment was expressed as

$$\beta_{bridge,i} = \min\left[\beta_1, \beta_2, \ldots, \beta_n\right] \tag{17}$$

An average reliability index for each bridge type, $\beta_{bridgetype}$, was then calculated in order to characterize the reliability level for that bridge type:

$$\beta_{bridgetype} = \frac{1}{5}\sum_{i=1}^{5} \beta_{bridge,i} \tag{18}$$

where the subscript *bridgetype* indicates the bridge type: steel beams (SC), prestressed I-beam (PI), prestressed adjacent box girders (PCA), or prestressed spread box girders (PCS). The β values under the summation sign were obtained from Equation 17. The subscript *i* indicates which of the five bridges within that bridge type.

The average reliability indices according to Equation 18 for the Metro Region are shown in Tables 2 and 3 for the as-designed and design-minimum strengths, respectively. As seen, average β for the 50th and 90th percentile ADTT values are listed separately, as well as for different load effects (moment or shear). Those reliability indices under 3.5 are shaded for comparison. As mentioned earlier, a target reliability index of 3.5 had been selected for this study to evaluate the adequacy of current design load for bridge design.

It is seen in Table 2 that the influence of the ADTT percentile values appears to be extremely small. For example, the reliability index β for FC 01 for shear in steel bridges is 7.0 for the 50th percentile traffic volume in Table 2. It is also 7.0 in the same table for the 90th percentile traffic volume. This situation is typical in Table 2. Such a negligible difference is due to the fact that the 50th and 90th percentiles are both fairly large, although different from each other. The projection performed using Equations 6 and 7 with a large exponent of *N* has resulted in very similar mean and standard deviation values for live load effect. This led to almost the same reliability index β values in Table 2 for different ADTT percentiles.

Comparison of as-designed and design-minimum cases in Table 2 shows that the over-design due to individual designers exercising conservatism is significant. For example, the β value for functional class 11 for shear in steel bridges was found to be 5.1 in Table 2 for the 50^{th} percentile of ADTT using the as-designed strength. In Table 2, the β value for the design-minimum strength for the same case is only 2.3, noticeably less than 5.1 and also below the target level of 3.5. To further demonstrate this, Figure 2 presents a graphical comparison of the design-minimum and as-designed reliability indices for a single case. This case is for moment at the governing cross section using the 50^{th} percentile ADTT for FC11 in the Metro Region. Note that these are bridge girder reliability indices and not the averages presented in Table 2 as defined by Equation 18. The largest difference in Figure 2 is observed in a prestressed adjacent box girder bridge, and the smallest difference is observed in a prestressed spread box girder bridge. The steel and concrete I-beam structures had approximately the same variation except one steel bridge's reliability index exceeded 5.

Tables 2 and 3 also show that β values for FC 11 and FC 12 consistently represent the worst conditions for the bridges in the Metro Region. Note that these two functional classes are defined as "urban principal arterial – interstate" and "urban principal arterial – other freeways or expressways", respectively, according to the FHWA definition. It is clear that the locality of truck loads was primarily responsible for the low levels of structural reliability.

It can also be seen in Table 2 that many of the prestressed concrete bridges analyzed in this study had reliability indices lower than 3.5 for the design-minimum cases. Actually, this was true for a majority of the cases analyzed in this study (6). While steel bridges have more values of β that exceeded 3.5, the lowest β value was seen for shear in a steel bridge span (S20-41064 listed in Table 1), equal to 1.2, but not explicitly shown in Table 2. This low β value was for both the 50^{th} and 90^{th} percentiles of traffic volume ADTT. The difference between the ADTT values is negligible as discussed above. These minimum β values are much lower than the target level of 3.5. Using the target β as a decision tool, it can be concluded that the minimum strength requirement in the current design code (2) does not provide the desired margin of safety, at least, for bridges in the Metro Region in Michigan.

It has been found that the reliability indices for the case of the entire state are very similar to those presented in Tables 2 and 3 for the Metro Region (6). The main reason for this is that both cases used the same WIM data collected from around the Metro Region (4), although the ADTT data was more specific for the analysis. Therefore, β values for the entire state are not presented here primarily because the WIM data was not gathered from a wide enough spectrum of sites throughout the state. Hence, the above conclusion should not be extrapolated to the entire state. This also suggests that more WIM data are needed to draw definitive conclusions for the rest of the state.

It also should be noted that the average β values for the as-designed strength in Table 2 are all above 3.5 except for shear in adjacent prestressed concrete box girder bridges. These lowest values occurred for FC11 and FC12 representing the worst loading condition among those considered here.

CONCLUSIONS

The following conclusions are reached based on the results of this study.

1. The difference in the structural reliability index β due to the traffic volumes taken as the 50^{th} and 90^{th} percentiles is negligible. The cases of "entire state" and "Metro Region" in this study did not result in significantly different β values, because the only difference between the two cases was the traffic volumes representing respective areas. Note that the WIM data (i.e., the probabilistic distribution of truck weight) was the same in the analyses for these two cases. These data were collected from bridges in or very near the Metro Region. No WIM data from other parts of the state were considered to be appropriate for use in this study.

2. Based on the twenty randomly selected bridges analyzed in this study, the current Michigan design load of HS25 for design of bridge beams did not consistently achieve the target reliability index of 3.5, especially for the design code-required minimum strength. Hence, the HS25 design load may be inadequate for the Metro Region. This conclusion was reached based on the following observations. i) A large number of cases of shear and moment for the 20 sample bridges used here had a β value lower than the 3.5 target level, particularly for FC11 and FC12 which are typical for the Metro Region. Note that this comparison is based on the design-minimum strength because the 3.5 target value is intended to be for this case. ii) Based on the as-designed strength, it has been found that there are still cases where the β value falls below 3.5. This indicates that convention or intentional conservatism by designers certainly mitigates the situation considerably. Nevertheless it cannot guarantee that all designs will produce bridge components with reliability indices above the target level. iii) The minimum β values averaged over each bridge type (Tables 2 and 3) also show similar trends as in items i) and ii). It also should be noted that these averaged minima were obtained over the bridges calculated here. They do not necessarily proportionally cover the entire bridge population in the Metro Region, because the sample bridges were not selected according to that population. On the other hand, it should be pointed out that it has not been established that a reliability index β value below 3.5 indicates an unsafe bridge. Particularly, the target level of 3.5 is used for single structural components and not the entire structural system. It is the system that dictates the safety of a bridge, not a single component. Secondly, the target level of 3.5 was selected in the calibration process of the AASHTO LRFD code as the average of β levels assured by the AASHTO standard design code at the time. Thus, it is not an absolute criterion but rather a relative norm.

3. More WIM truck weight data beyond what have been used in this study are needed to investigate whether the above conclusions are also valid for other regions in the state. The WIM datasets used in this study were collected from bridges in or very near the Metro Region. These data were used for reliability analysis for other regions in the state but with traffic volumes from the corresponding areas. Resulting β values are very similar to those for the Metro Region, as indicated in item 1 above. It is seen that the same WIM data used for both the "entire state" and the "Metro region" cases makes it difficult to draw reliable conclusions as to the entire state. Therefore, WIM data from other regions are required to draw definitive conclusions for the rest of the state.

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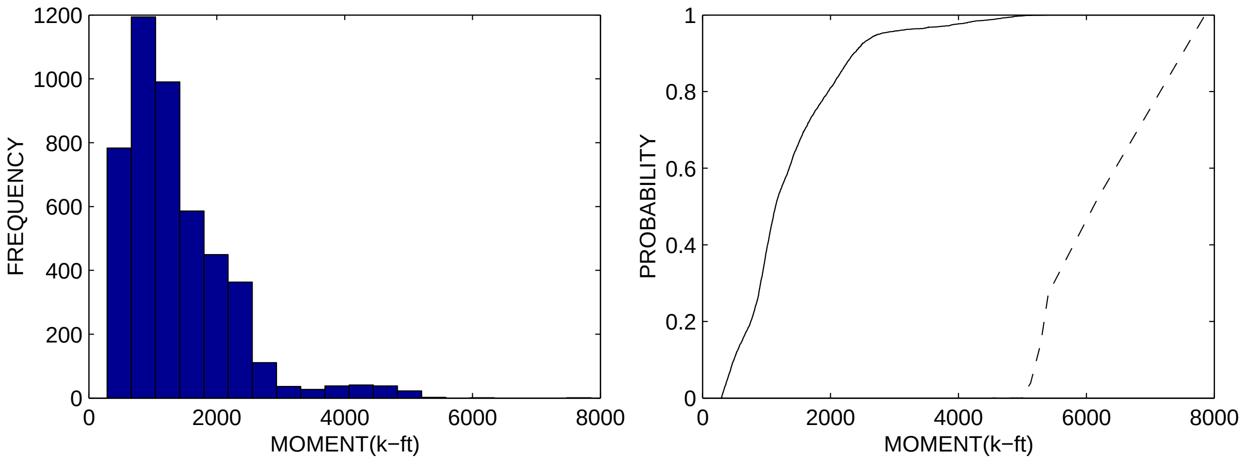

FIGURE 1 Example of histogram for truck-induced moment, empirical cumulative distribution function (solid line), and cumulative distribution function projected to 75 years (dashed line).

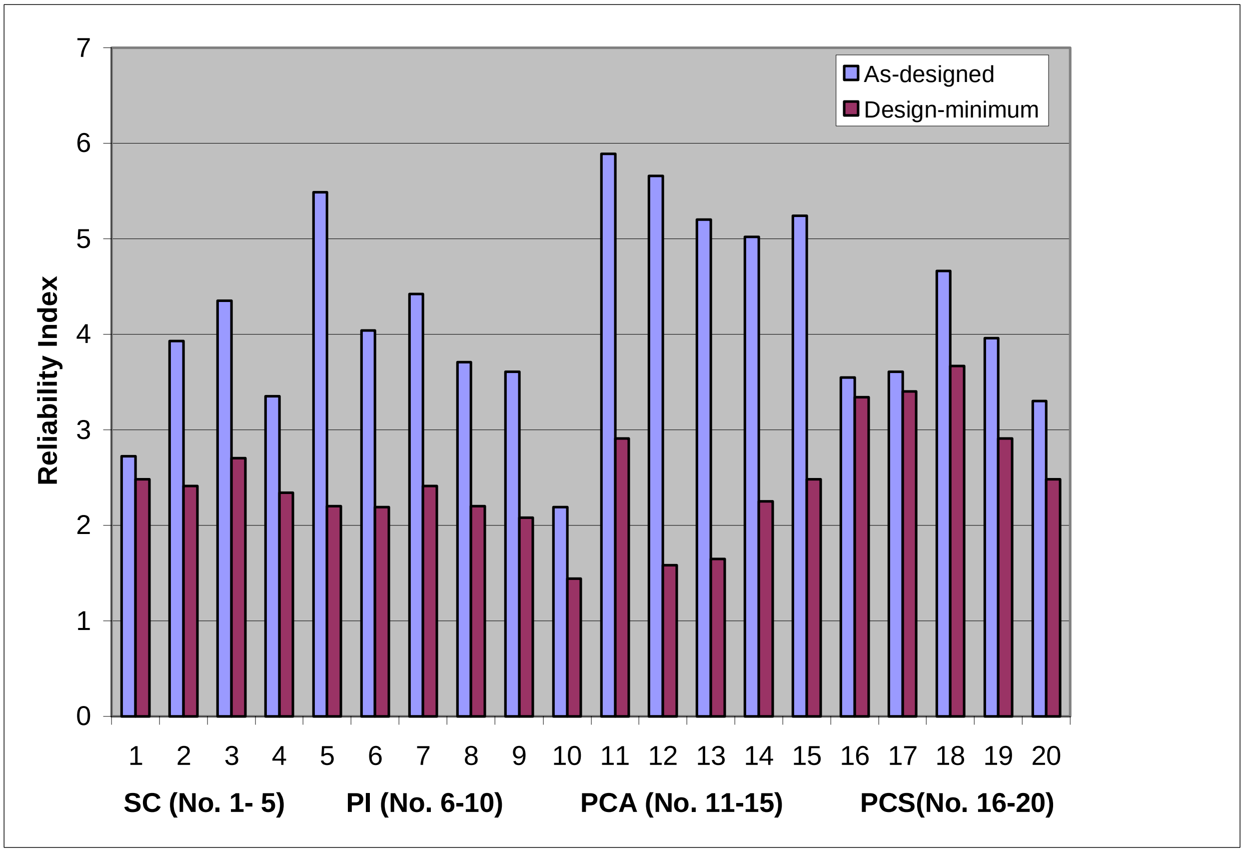

FIGURE 2 Comparison of Design-minimum and As-designed Reliability Indices for the 50th percentile ADTT in Metro Region for FC11 (moment)

TABLE 1 Details of Bridges Used in Reliability Analysis

Bridge Type and I.D.	**Number of Spans**	**Span Length** (ft./meters) {span ID}	**Span Type**
SC[1]			
11072-B01	2	66 / 20.1 {1 & 2}	Continuous
19042-S03	4	151 / 46 {1}, 127.2 / 38.8 {2 & 3}	Continuous
41064-S20-3	1	130.6 / 39.8	Simply Supported
41064-S18	1	146 / 44.5	Simply Supported
63174-S19	2	145.3 / 44.3 {1}, 160.8 / 49 {2}	Continuous
PI[2]			
19033-S11	1	129 / 39.3	Simply Supported
11112-B02	7	118.5 / 36.1 {1 & 7}, 116.3 / 35.5 {2,3, & 6}, 116.8 / 35.6 {4 & 5}	Simply Supported
11052-B02	4	98 / 29.9 {1}, 98.4 / 30 {2,3 & 4}	Simply Supported
19034-R01	3	41 / 12.5 {1 & 3}, 32.5 {2}	Simply Supported
11057-B04	7	123.8 / 37.7 {1 & 7}, 123 / 37.5 {3,4,5 & 6}	Simply Supported
PCA[3]			
46082-B02	1	41.3 / 12.6	Simply Supported
82022-S05	2	71.5 / 21.8 {1 & 2}	Simply Supported
82022-S06	2	71.5 {1 & 2}	Simply Supported
82022-S25	1	95.6 / 29.1	Simply Supported
11015-S01	4	36.4 / 11.1 {1}, 76.8 / 23.4 {2 & 3}, 41.9 / 12.8 {4}	Simply Supported
PCS[4]			
33084-S14	3	38.4 / 11.7 {1}, 70.6 {2}, 34.9 / 10.6 {3}	Simply Supported
55011-R01	1	72.8	Simply Supported
63081-S06	3	28.1 / 8.6 {1}, 73.8 / 22.5 {2}, 29 / 8.8 {3}	Simply Supported
79031-B01	1	46.6 / 14.2	Simply Supported
03072-B04	1	52 / 15.9	Simply Supported

[1]SC = Steel Beams with Concrete Deck
[2] PI = Prestressed Concrete I-Beams
[3] PCA = Prestressed Concrete Adjacent Box Girders
[4]PCS = Prestressed Concrete Spread Box Girders

TABLE 2 Reliability Indices β for As-designed and Design-minimum Girder Capacity for Metro Region in Michigan

As-designed		Functional Class[1]									
Bridge Type		01		02		11		12		14	
		50th	90th	50th	90th	50th	90th	50th	90th	50th	90th
SC	shear	7.0	7.0			5.1	5.0	4.9	4.8	7.4	7.4
	moment	5.5	5.5			3.9	3.9	3.9	3.8	5.7	5.7
PI	shear	8.1	8.1			6.6	6.5	6.5	6.4	8.3	8.2
	moment	5.8	5.8			3.6	3.5	3.6	3.5	6.0	6.0
PCS	shear	8.2	8.2			6.3	6.2	6.3	6.3	8.3	8.2
	moment	8.3	8.3			5.4	5.4	5.8	5.7	8.4	8.4
PCA	shear	4.9	4.9			2.9	2.9	3.1	3.1	5.0	4.9
	moment	6.1	6.1			3.8	3.8	4.0	4.0	6.2	6.2
Design-minimum		Functional Class[1]									
Bridge Type		01		02		11		12		14	
		50th	90th	50th	90th	50th	90th	50th	90th	50th	90th
SC	shear	4.3	4.3			2.3	2.3	2.2	2.1	4.6	4.6
	moment	4.1	4.1			2.4	2.4	2.4	2.3	4.4	4.7
PI	shear	3.5	3.5			2.0	2.0	2.0	1.9	3.6	3.6
	moment	4.3	4.3			2.1	2.0	2.1	2.0	4.5	4.5
PCS	shear	3.8	3.8			2.0	1.9	2.0	2.0	3.8	3.8
	moment	5.0	5.0			2.2	2.1	2.6	2.5	5.1	5.1
PCA	shear	3.1	3.1			1.2	1.2	1.4	1.4	3.2	3.2
	moment	5.6	5.6			3.2	3.1	3.5	3.4	5.7	5.7

[1]FC01 = Principal Arterial – Interstate Rural
FC02 = Principal Arterial – Other Urban
FC11 = Principal Arterial – Interstate Urban
FC12 = Principal Arterial – Urban
FC14 = Other Principal Arterial - Urban