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SYSTEM RELIABILITY MODELS FOR BRIDGES *

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ABSTRACT

There is a need for efficient bridge evaluation procedures. A considerable number of existing bridges in the United States require repair and/or replacement. Accurate estimate of the current strength and the remaining life time is essential for optimum distribution of the available limited resources. The major parameters which determine the structural performance are random variables. Statistical models of load and resistance can be derived from the test data, observations and analysis. For bridge members the reliability can be calculated using this available data. However, there is a considerable discrepancy between the reliability level of individual members and the overall bridge reliability. Due to load sharing and redundancies, the actual load-carrying capacity often exceeds the theoretical value. System reliability methods allow us to reveal an actual safety reserve in the structure. This paper summarizes the practical bridge reliability models. Using a special sampling technique, the reliability is evaluated for typical girder bridges.

1. INTRODUCTION

The evaluation of bridges is an increasingly important topic in the effort to deal with the deteriorating infrastructure in the United States. About 40% of the nation's 570,000 bridges are classified, according to the Federal Highway Administration's (FHWA) criteria, as deficient and in need of rehabilitation and replacement [1]. In other countries the situation is similar. The major factors that have contributed to the present situation are: the age, inadequate maintenance, increasing load spectra and environmental contamination. The deficient bridges are posted, repaired or replaced. The disposition of bridges involves clear economic and safety

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implications. To avoid the high costs of replacement or repair, the evaluation must accurately reveal the present load-carrying capacity of the structure and predict loads and any further changes in the capacity (deterioration) in the applicable time span.

The accuracy of bridge evaluation can be improved by using the recent developments in bridge diagnostics, structural tests, material tests, structural analysis and probabilistic methods. Advanced diagnostic procedures can be applied to evaluation of the current capacity of the structure, monitoring of load and resistance history and evaluation of the accumulated damage. Full-scale bridge tests provide very useful information about structural behavior. There is a need for significantly more test data, covering various bridge types. However, extensive test programs are very costly. Therefore, considerable effort should be directed towards evaluation and improvement of current analytical methods, on the basis of available test data.

The recent developments on the area of materials allow for a better understanding of the behavior of bridge components. The availablity of new materials may improve bridge repair techniques. More data are now available on bridge loads, static and dynamic. Computer methods for structural analysis have improved the accuracy of representing the acutal load-sharing behavior of bridge components. Available diagnostic technologies, including state-of-the-art digital signal processing, integrated sensor systems, and microwave, acoustic and optical techniques, can find application in bridge evaluation.

New advances that are significant for bridge evaluation have taken place in the area of probabilistic methods. Reliability has become an issue of increasing importance for highway bridges because of gradual increases in vehicle loads, difficulties of law enforcement and increase in traffic. More exact methods of analysis that leave less generous safety margins over and above the minimum required by specifications, combine with more deleterious service environment and limitations on maintenance to reduce the reliability of bridges. A probabilistic treatment is appropriate to account for the influence of these trends.

The objective of this paper is to present an approach to evaluation of the reliability for typical girder bridges. Load and resistance models for structural members are derived from the available data. Bridge reliability is calculated using a special sampling technique.

2. LIMIT STATES IN BRIDGES

A structure fails when it can no longer perform its function. The function includes not only structural aspects such as to carry safely traffic loads for expected periods of time and to provide a comfortable passage (minimize deflections and vibrations), but also to provide aesthetic values to the users. The structural performance can usually be expressed in terms of mathematical equations and limit state functions, involving various parameters of material properties, dimensions and geometry. In the simplest case, let Q be the load effect in a structural member and R be the load-carrying capacity of the member. Then, the structural failure corresponds to R being less than Q. The corresponding limit state function, g, is

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

The negative values of g indicate failure, so that the probability of failure, P_F, is

$$P_F = P(g < 0) \tag{2}$$

where $P(\ ) =$ probability of event $(\ )$.

In practice, a simple formulation of the limit state function is often impossible. Both resistance, R, and load effect, Q, are functions of multiple parameters. These parameters vary randomly, some are difficult to quantify (e.g. geometric configuration), they are often time-variant (e.g. live load or effect of corrosion) and/or correlated. Some bridge limit states require special formulations. For example, fatigue, concrete cracking, corrosion effects, vibrations or deflections. Typically, the performance of a structural member is described by a set of limit state functions.

For a typical bridge girder, the limit state functions can be formulated for various conditions, including:

- Bending moment capacity; R can be defined as the moment-carrying capacity for the girder, and Q as the maximum load resisted by the girder. Q is a joint effect of dead load, live load, environmental effects, and so on.
- Shear capacity.
- Buckling capacity; overall and local.
- Deflection; excessive deformations may affect the user's comfort or cause some structural problems (e.g. increased dynamic truck load).
- Vibrations; may affect the user's comfort.
- Accumulated damage conditions, including corrosion (rebars in reinforced concrete beams and slabs, pretressing tendons, structural steel sections), fatigue (mostly affects steel sections, prestressing tendons and rebars), cracking (concrete) and other forms of material deterioration.

Each limit state is associated with a set of limit state functions which determines the boundaries of the acceptable performance. For the accumulated damage, eqn. (1) is not adequate. Load effect, Q, is not a single force but a series of forces corresponding to the truck axles passing over the bridge. The capacity, R, requires a special formulation. For a given traffic density ADTT (average daily truck traffic) and cumulative distribution functions of axles weights, R can be expressed in terms of the number of years to failure (expected life time).

In general, bridge members are interconnected and they share the loading. Load distribution is not necessarily linear and it may vary depending on load magnitude, point of application or degree of deterioration. Limit states for the whole structure, as for individual members, express the boundary lines between safe (acceptable) realizations and failure. The critical conditions for bridges can be defined in terms of maximum deformations (e.g. deflections), vibrations, or the condition of individual members (girders and slabs). Usually, reaching a limit state by one of the members does not mean that the bridge also reached its limit state. Traditional design and analysis of bridges is based on identification of the governing limit state in each member. Safety provisions are applied to ensure an adequately low probability of occurrence for these limit states. In many bridges this approach is excessively conservative. Yet, in the evaluation of existing structures there is a need for an accurate estimate of capacity. The tool for calculation of the reliability for the whole bridge is provided by the theory of system reliability.

For example, a girder bridge can be represented by a system of longitudinal elements (girders). Live load is distributed to the girders through a flexible medium (slab and diaphragms). In general, girders, at least groups of adjacent girders, can be considered as parallel elements in the system reliability. Bridge behavior depends on the degree of correlation between strengths of girders (correlation reduces the system reliability).

3. SAFETY RESERVE FOR BRIDGES

Numerous bridge tests indicate a considerable difference between the predicted (by analysis) capacity and the minimum capacity determined by proof loading [2]. Many bridges, posted for reduced load, show a strength level adequate to carry the regular traffic. The major factors responsible for the discrepancy between analysis and practice are load sharing and redundancy. Load sharing is the distribution of forces caused by traffic and other load components on structural and nonstructural members. The effect of the latter is usually neglected. The connections or supports often behave differently than in theory. On the other hand, redundancy is the ability of the structure to continue to function safely in an almost normal manner despite the failure of one of the main load-carrying elements.

To evaluate the redundancy of a structure, the failure modes of the main load-carrying members must be examined to determine the possible secondary (redundant) load paths. These load paths must then be evaluated to determine that there are no weak links that would prevent the development of their full capacity. Studies can be made of various typical types of structures that would simplify the checks for redundance, e.g. design criteria could be developed for bridge deck slabs which are a key element in the redundancy of stringer bridges. Design criteria can also be developed for checking the ability of the adjacent stringers to carry a "failed" stringer. Assurance of this ability of a structure could reasonably allow the safety reserve of an individual member to be reduced.

There are at least two load paths in a redundant structure: a primary path and a secondary path. However, the secondary path should not be equally as stiff as the primary path. There must be some indication that the bridge is in distress. If not, the bridge may continue to be used without repair until the secondary path also fails.

Redundancy must be clearly separated from the natural interaction of bridge members that is often underevaluated in the design process. In the past, the use of the load distribution factor generally resulted in an underevaluation. Using newer methods of analysis a much more accurate evaluation can be made. In the result the underevaluation of the interaction of a bridge designed using an analysis method which propertly considers all structural members is not significant for the ultimate strength of the bridge. In serviceability conditions the interaction of the nonstructural elements, such as wearing surface, curbs, parapets, etc., may still have a significant role. The analysis and quantification of redundancy in bridge structures can be done using the system reliability methods.

4. LOAD AND RESISTANCE MODELS FOR BRIDGES

The major load components of highway bridges are dead load, live load with impact, environmental loads, earth pressure, and abnormal loads. Each load group includes several subcomponents. The statistical models for the various bridge loads can be based on the available data and on the results from special studies.

Dead load, D, is the gravity load due to the self-weight of the structural and nonstructural elements permanently connected to the bridge. Because of different degrees of variation, it is convenient to consider four components of D; D_1, weight of factory-made elements (steel, precast concrete members); D_2, weight of cast-in-place concrete members; D_3, weight of the wearing surface (asphalt); and D_4, miscellaneous weight (e.g. railings, luminaries). The distribution of all components of D can be taken as normal. The bias (mean-to-nominal ratio) varies; for D_1 it is 1.03, and for D_2 it is 1.05. The coefficients of variation are assumed to be 0.04 and 0.08 for D_1 and D_2, respectively. However, $V_D = 0.10$ is recommended for D_1 and D_2 because of analysis uncertainties. Other components of dead load include weight of asphalt, railings, curbs, luminaries, signs, conduits, pipes, cables, etc. The coefficient of variation for the thickness of asphalt is 0.25 [3]. Other statistical parameters (biases and coefficients of variation) are similar to those of D_1 if the considered item is factory-made with the high quality control measures, and D_2 if the item is cast-in-place, with less strict quality control.

Live load, L, covers a range of forces produced by vehicles moving on the bridge. The effect of live load depends on wheel force, wheel geometry (configuration), position of the vehicle on the bridge (transverse and longitudinal), number of vehicles on the bridge (multiple presence), stiffness of the deck (slab), and stiffness of the girders. The live load model, based on weight-in-motion (WIM) data, was developed by Ghosn and Moses [4]. The maximum 50-year bending moment in the girders is calculated as a function of truck configuration, span length, truck type, 95th percentile of weight for the dominating truck type at the site, probability of closely spaced vehicles on a bridge (this accounts for both the multiple presence likelihood and the likelihood that truck weight exceeds the characteristic 95th percentile truck weight value), girder distribution factor, dynamic amplification factor, and future growth factor. For example, the bias factor of L varies from 1.09 to 1.34 and the coefficient of variation is 0.21 to 0.28 for spans from 12 to 30 m.

The environmental loads include wind, earthquake, snow, ice, temperature, water pressure, etc. The load models can be based on the report by Ellingwood et al. [5]. The basic data have been gathered for building structures, rather than bridges. However, in most cases the same model can be used. Some special bridge-related problems can occur because of the unique design conditions, such as foundation conditions, extremely long spans, or wind exposure.

Bridge load models require further research. For example, further studies are needed to establish the live load model for various sites, time periods, distribution of truck weight, multiple presence in one lane and in adjacent lanes, and the transverse positions of trucks on the bridge. An additional area in need of further research is dynamic load. Further field measurements are required to determine the effect of structural type and surface condition. Additional studies must be carried out in order to estimate the growth in future live loads.

The resistance of bridge members, R, is a random variable. It is convenient to consider R as a function of three variables M, F, and P

$$R = R_n MFP \tag{3}$$

where R_n is the nominal (design) value of resistance; M represents material properties, including strength, modulus of elasticity, cracking stress, chemical composition; F represents fabrication effects, mostly geometry, dimensions, section moduli; and P is the analysis factor, including uncertainties due to approximate analysis, idealized stress and strain models, support conditions.

Some information about variation of the parameters M, F, and P is available for the basic structural materials, members and connections. However, bridge members are often made of several materials (composite sections) which may require special methods of analysis. Verification of the analytical model can be very expensive because of the large size of bridge members. Extensive tests of material properties and member behavior were performed for building structures. The available data is summarized by Nowak et al. [6]. For example, for steel girders the bias factor of R is about 1.01 and $V_R = 0.12$; for reinforced concrete the bias factor is

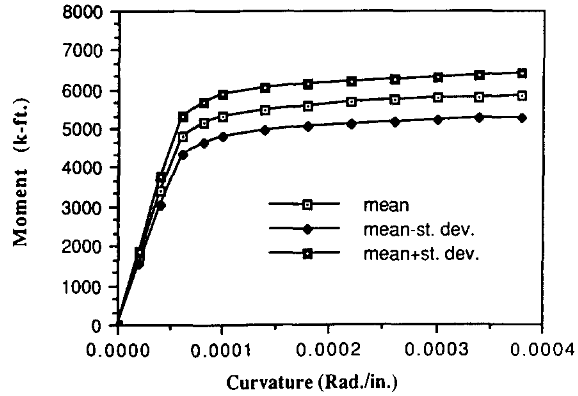

Fig. 1. Moment–curvature relationship for a W36×300 composite steel girder. Units: 1 in. = 25 mm; 1 k-ft = 1.356 kN-m.

1.20–1.25 and $V_R = 0.11$; and for prestressed concrete girders the bias factor is 1.05 and $V_R = 0.06$.

The material properties and dimensions of bridge members vary randomly. Therefore, the load–deformation relationships also vary. Example of a moment–curvature curve is shown in Fig. 1 for a typical composite steel girder [7].

5. BRIDGE RELIABILITY ANALYSIS

The major parameters considered in this analysis are moment–curvature curves for the girders (one curve for each girder) and the weight and position of trucks on the bridge. Moment–curvature curves were simulated. Two live load cases were considered: a single truck and two trucks side-by-side. Trucks were placed along the span to maximize the bending moment. Several transverse positions were considered (weighting factors were applied depending on traffic frequency corresponding to each position). Truck load was increased gradually, until the ultimate load was reached.

The sampling techniques described by Zhou [8] were used. The parameters of resistance were calculated using integration sampling technique. Specific point and weights are predetermined in the independent standard normal space. The sample points in basic variable space are then obtained by various transformations. Truck type (axle configuration) and truck position (curb distance and multiple presence) are represented by additional weighting factors. These simulations provided the parameters of bridge resistance.

The reliability indices were calculated for typical bridges, one with steel girders and the other with prestressed concrete girders. The steel bridge considered is a simply supported structure, with 18-m span and the cross-section shown in Fig. 2. The prestressed concrete bridge, with a simple span of 27 m, is shown in Fig. 3.

Various degrees of correlation between the capacities of adjacent girders were considered. The reliability indices, calculated for the structural system, vary from 6.05 (perfect correlation) to 6.95 (no correlation) for the steel bridge, and from 5.00 (perfect correlation) to 5.25 (no correlation) for the prestressed concrete bridge [9]. These reliability indices can be compared

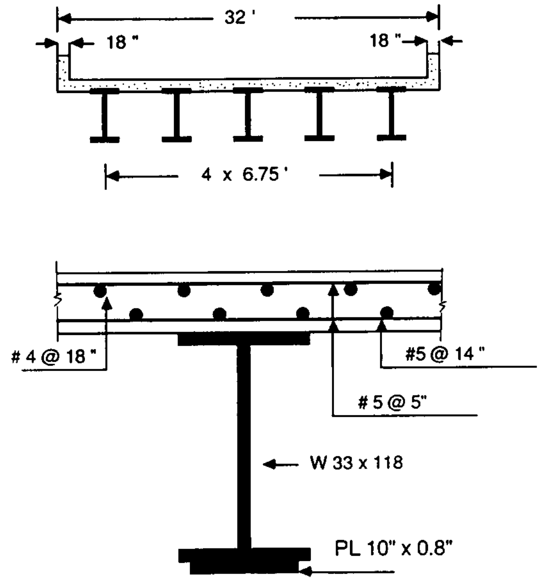

Fig. 2. Composite steel girder bridge considered in the analysis. Units: 1 in. = 25 mm; 1 ft = 305 mm.

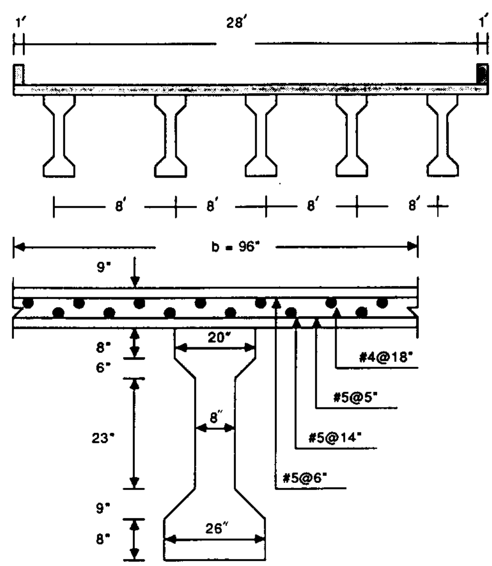

Fig. 3. Prestressed concrete girder bridge considered in the analysis. Units: 1 in. = 25 mm; 1 ft = 305 mm.

with the reliability indices calculated for individual girders: 3.35 for a steel girder and 4.85 for a prestressed concrete girder [6].

6. SUMMARY AND CONCLUSIONS

Reliability indices were calculated for typical bridges, one with composite steel girders and the other with prestressed concrete girders. The calculations were performed for various degrees of correlation between the girders. Little effect of correlation was observed.

The results were compared with the reliability indices calculated for individual girders. A considerable difference between the element and system reliabilities can be attributed to the redundancy and load sharing in a girder bridge.

System reliability methods provide an efficient tool for evaluation of performance of existing bridges.

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