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Reliability-based design optimization of a vehicular live load model

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ABSTRACT

Typically, RBDO is used to select physical characteristics of a structure in order to optimize performance under uncertainty, where design variables are often taken as geometric or material parameters defining the structure. In this study, the effectiveness of using RBDO to develop a design load model is explored, which is to be used to design a wide range of bridge girders subjected to location-specific traffic loads. The objective is to minimize variation in reliability index among different bridge girders designed using the load model. Design variables are taken as the number of axles, axle spacing, and axle weights of the design vehicle representing the live load model. A probabilistic constraint is imposed, limiting the minimum reliability index of each girder design. Random variables considered are girder resistance in moment and shear, and bridge dead load and vehicular live load components. A single loop procedure is implemented by using a non-iterative, modified reliability method, the final results of which are verified with direct Monte Carlo Simulation. The optimization problem is solved with a genetic algorithm. It was found that procedure employed could be used to develop a design load model that resulted in substantial improvement over the design models in current use, where the optimized models significantly reduced the range and coefficient of variation of reliability index among the bridge cases considered.

1. Introduction

Bridge design in the US is governed by the AASHTO LRFD Bridge Design Specifications [1], which was first published in Load and Resistance Factor Design format in 1994. Using a reliability-based calibration approach, the purpose of the LRFD version was to provide a more consistent level of safety than that available under the existing Standard Specifications, which were last revised in 2002 [2]. A major part of the LRFD calibration effort was to revise the vehicular design load model. Although extensive weigh-in-motion (WIM) data exists for major roads throughout much of the US today, very limited data were available at the time of the 1994 AASHTO code formation. As a result, the code calibration relied upon a small sample of less than 10,000 heavy truck weights recorded in Ontario, CA, in 1975. The load effects caused by this sample, which represented two weeks of data, were extrapolated to a 75 year period of time to generate the required live load random variable statistics needed for reliability analysis. This database contained only single vehicle entries, and no information on side-by-side or following vehicles was available. As such, several assumptions as well as simulations were used to estimate simultaneous presence load effects; for example, that every 15th heavy vehicle was side-by-side with another, and that every 30 side-by-side events were fully correlated with regard to weight. The reliability analysis that incorporated this modeling effort was used to develop the HL-93 design load within AASHTO LRFD, where live load and dead load factors were chosen to allow bridge girders to meet a minimum notional target reliability index of 3.5 [3].

As WIM data collection became more common, various state departments of transportation (DOT) recognized the potential mismatch between the limited vehicular data used to develop the LRFD design load model and the actual traffic loads experienced in their states. As such, multiple DOTs initiated state-specific live load model development efforts to refine the LRFD design model to local traffic conditions. Some of these efforts were guided by the release of NCHRP Report 683 [4], which offered detailed guidelines to collect WIM data, project load effects, and conduct a reliability-based design calibration for live load factors. The report authors used this procedure to develop a new live load model to rate bridges in New York [5]. Related efforts include that by Kwon et al. [6] and Pelphrey et al. [7], who recalibrated live load factors for bridge design and rating for the Missouri and Oregon State DOTs, respectively. Although not complete recalibration studies, Tatabai et al. [8] characterized maximum load effects for Wisconsin DOT by fitting multi-modal distributions to axle loads and spacings collected from WIM data, then using simulation to model the axle load and

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spacing relationships. Earlier, Lee and Souny-Slitine [9] modeled equivalent single axle loads from WIM data for Texas DOT.

The complexity of adjusting design loads to better correspond to load effects can be well-illustrated by the effort over time of the State of Michigan. Before the availability of WIM data, Michigan DOT (MDOT) first increased its design load beyond that specified by AASHTO in 1973, in recognition that the traffic loads allowed in the State were generally larger than those assumed in the AASHTO standard [10]. Several decades later, once WIM data became readily available in the State, several research projects were conducted to quantify actual State traffic loads [11]. At the time of these studies, however, Michigan WIM data were collected at low fidelity, where only single vehicle information could be captured, neglecting the often-governing configurations of following or side-by-side vehicle groups. Using various approximations somewhat similar to those in the AASHTO LRFD calibration to simulate the effects of multiple vehicle presence, these research efforts produced the design load currently used by MDOT, referred to as HL-93-mod. This design load is substantially higher than the AASHTO LRFD HL-93 design load for some bridge spans.

Most recently, Eamon et al. [12] evaluated the effectiveness of the HL-93-mod design load, in conjunction with higher quality WIM data as well as a wider range of bridge geometries than originally considered. This evaluation found significant variation in the required live load factor among different bridge geometries to produce uniform levels of reliability, where in some cases, a particular structure required twice the live load factor as another. It was also found that a significant proportion of structures had insufficient levels of safety, where typical composite steel and prestressed concrete girder bridges spanning from 80 to 200 ft had reliability indices between 2 and 3, under the desired target of 3.5. However, the pattern of required load factors was complex, varying with bridge geometry, failure mode, and girder type, with no obvious live load model well-fitting results. Although increasing the load factor on the existing HL-93-mod load to the minimum required to cover every case is a possible solution, this would result in substantial overdesign in the majority of structures. Thus, an alternative procedure is desired to develop an appropriate live load model. One possible process explored in this study is the use of reliability based design optimization (RBDO).

Typically, RBDO is applied to select the physical characteristics of a structure, represented with design variables, to optimize performance under uncertainties. This approach has been used to minimize initial as well as life cycle costs of bridge structures [13–17], as well as to minimize the weight of bridge components [18–20]. RBDO has also been applied to unique bridge structures, including suspension [21] and cable-stayed bridges [22], as well as to produce optimal designs under extreme loads [23]. There are multiple ways of formulating and solving an RBDO problem [24–29, etc.]. Different approaches for evaluating probabilistic constraints have been developed, primarily in an attempt to reduce computational effort [24,26]. For example, Du and Chen [30] suggested to replace probabilistic constraints with equivalent deterministic limits, to allow decoupling of the reliability analysis and optimization algorithms in each design cycle. Similarly, Qu and Haftka [31] proposed utilizing equivalent safety factors to represent probabilistic constraints.

In this study, however, design variables (DVs) are not taken as geometric characteristics of structural components, as in the traditional RBDO approach. Rather than directly optimizing a structural component, DVs are taken as characteristics of the process used for design, and in particular, the live load model. The objective of this study is thus to apply the RBDO concept to the design process itself, such that an optimal live load model can be developed for bridge structural design. In this approach, optimum notional design vehicle configurations are developed such that when used, inconsistencies in reliability among different types of structures are minimized.

2. Reliability-based design optimization

Probability theory is most commonly used to model uncertainty in design optimization. This combination of probabilistic modeling and mathematical design optimization is incorporated within RBDO. In RBDO, uncertainty associated with load intensity, material properties, geometric dimensions, and other parameters can be represented by a vector of random variables $\boldsymbol{X} = \{X_1, X_2, ..., X_n\}^T$ and propagated by mathematical models that quantify variability in responses as functions of such random variables. An RBDO problem is also described with a set of design variables $\boldsymbol{Y} = \{Y_1, Y_2, ..., Y_{NDV}\}^T$. Often, random variables $\boldsymbol{X}$ and design variables $\boldsymbol{Y}$ overlap, where the mean values of random variables are commonly design variable values. If $g(\boldsymbol{X},\boldsymbol{Y})$ represents a stochastic response function for measuring structural performance, then a failure condition can be defined as $g(\boldsymbol{X},\boldsymbol{Y}) \leqslant 0$, whereas $g(\boldsymbol{X},\boldsymbol{Y}) > 0$ implies safety with $g(\boldsymbol{X},\boldsymbol{Y}) = 0$ representing the limit-state boundary that separates the safe and failure regions of random variable space. For this response function, the probability of failure is defined as the probability of $g(\boldsymbol{X},\boldsymbol{Y}) \leqslant 0$.

In its generic form, an RBDO problem typically seeks to minimize an objective function subject to a series of probabilistic and possibly deterministic constraints. For this study, the goal of the optimization is to develop a live load design model that can be used to produce structural designs of girders for bridges with differing geometry, material type, and failure mode that are as close as possible to the target reliability level, without falling below an acceptable minimum. In other words, assuming the minimum required reliability level is met, the variation in reliability among girders of different structures from the target level is to be minimized. The optimization problem can be thus formulated as:

$$\begin{aligned} \min \quad & f(\boldsymbol{X},\boldsymbol{Y}) \\ \text{s.t.} \quad & P_{f_i} = P[g_i(\boldsymbol{X},\boldsymbol{Y}) \leqslant 0] \leqslant P_{ai}; \quad i = 1, N_p \\ & Y_k^l \leqslant Y_k \leqslant Y_k^u; \quad k = 1, NDV \end{aligned} \tag{1}$$

where $f(\boldsymbol{X}, \boldsymbol{Y})$ is an objective function representing variability in structural reliability among the different bridge girder designs considered (detailed below); P_{f_i} is the failure probability associated with a girder from bridge structure i among N_p structures considered; P_{ai} represents the allowable value or upper bound on the i^{th} failure probability; and Y_k is the k^{th} design variable among NDV design variables, with lower and upper bounds (side constraints) of Y_k^l and Y_k^u, respectively.

Utilizing reliability index (β) directly rather than failure probability, as commonly done in structural reliability analysis, the problem can be written as:

$$\begin{aligned} \min \quad & f(\boldsymbol{X},\boldsymbol{Y}) \\ \text{s.t.} \quad & \beta_i \geqslant \beta_{\min}; \quad i = 1, N_p \\ & Y_k^l \leqslant Y_k \leqslant Y_k^u; \quad k = 1, NDV \end{aligned} \tag{2}$$

where β_i is the reliability index constraint for girder i and β_{min} is the minimum acceptable reliability index. As noted above, the objective function must quantify variation in reliability among girders relative to the target level (β_T). That is, if all bridge girder designs exactly meet the desired reliability index, variation from the target is zero and an ideal solution is obtained. Numerous ways to quantify variation in this manner are available, such as mean absolute error, root mean squared error, R-squared, mean squared error, least absolute value, etc. In this study, mean squared error (MSE) is used, whereby the objective function is formulated as:

$$f(X,Y) = \sum_{i=1}^{N_p} \frac{(\beta_i - \beta_T)^2}{N_p} \tag{3}$$