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# Reliability evaluation of axially loaded steel members design criteria in AASHTO LRFD bridge design code

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ABSTRACT

Reliability based structural design insures a uniformly designed structure, in terms of safety. By considering an adequate reliability index (or probability of failure) for different parts of a structure, a reasonable balance between cost and safety of the structure can be achieved. In this study, the reliability of steel tension and compression members designed with AASHTO LRFD bridge design specifications (2007) is evaluated. These members are prevalent in different types of truss or arch bridges. Various conditions such as redundancy, ductility and importance of the bridge are taken into account by changing the load modification factor, $\eta$. To include the effect of the span length, a variable ratio of dead load to total load is considered. Current load factors in AASHTO LRFD code are accepted due to their verification in a comprehensive study for reliability of girder-type bridges. Furthermore, load and resistance distribution models are chosen based on the latest existing experimental data. The Monte Carlo simulation technique with randomly generated samples is applied in numerical calculations. For tension members, analysis results show relatively high reliability indices in yielding design, while having slightly low reliabilities for the fracture mode. For fracture design of steel tension members, an increase in vehicular dynamic load allowance (IM) from 33% to 75% is suggested to insure a safer behavior. Also, it is shown that the resistance factor for yielding of gross section, $\varphi_y$, can be increased from 0.95 to 1.00 while maintaining enough safety for designed tension members. In addition, obtained reliability curves for steel compression members show a safe behavior of designed compression members with conservative response in some cases. More results and plotted curves are discussed in detail and possible adjustments in code criteria are presented in this paper.



## 1. Introduction

Providing a reasonable balance between cost and safety of a structure has always been the major concern in developing design codes and specifications. A conservative design will enhance structural safety along with increasing cost of the construction. Converting all significant terms to an equivalent cost value including failure of the structure – product of the probability of failure and damage cost due to the failure – the final cost should be minimized to obtain the most optimum design.

The re-calibration of existing design criteria, including reliability-based ones, is unavoidable due to numerous technical improvements and changes in the cost factors. As an example, the application of fast computers in numerical calculations may increase the precision of analysis results and reduce human errors in design procedure. Moreover, material quality enhancement can reduce structural component imperfection, and subsequently probability of failure.

Additionally, load characteristics may change with time for each specific structure. For instance, more restrictive traffic rules may reduce the probability of overweight trucks passing on bridges. In fact, the latest dependable experimental data for both load and resistant parameters should be considered for any re-evaluation of the design criteria. However, simplification of design equations offers more conservative criteria in most cases.

A summary of various reliability studies, utilized as the backbone of the LRFD Bridge Design Code [1], is provided in NCHRP-368 [2]. Examining four different types of bridges with reinforced concrete girders, prestressed concrete girders, and composite and non-composite steel girders, as the most typical solutions in designing bridge structures, load and resistant factors were recalibrated to current factors. However, other types of bridge components such as axial members in trusses were not covered in the recalibration procedure [3].

Bennett and Najem-Clarke [4] evaluated the reliability of bolted steel tension members designed according to the AISC LRFD steel design code. Considering two failure modes; yielding of the gross

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**Nomenclature**

| | |
|---|---|
| $A_g$ | gross cross-sectional area |
| $A_n$ | net section area |
| DC | effect due to dead load |
| DW | wearing surface load |
| $E$ | modulus of elasticity |
| $F_y$ | specified minimum yield strength |
| $F_u$ | specified minimum tensile strength |
| IM | dynamic load factor |
| $K$ | effective length factor |
| $l$ | unbraced length |
| LL | effect due to live load |
| $P_f$ | probability of failure |
| $P_r$ | factored compressive resistance |
| $U$ | reduction factor to account for shear lag |
| $V$ | coefficient. of variation |
| $r'$ | dead load to total load ratio |
| $r_s$ | radius of gyration about the plane of buckling |
| $\beta$ | reliability index |
| $\beta_T$ | target reliability index |
| $\delta$ | bias factor |
| $\varphi_c$ | resistance factor for compression |
| $\varphi_y$ | resistance factor for yielding on gross section of tension member |
| $\varphi_u$ | resistance factor for fracture on net section of tension member |
| $\lambda$ | slenderness related parameter |
| $\eta$ | load modifier related to ductility, redundancy and operational importance |

section and fracture of the net section, the reliability index for each mode and combined system based on the correlation coefficient between yielding strength, $F_y$, and fracture strength, $F_u$, was derived. Based on their study, it can be concluded that for different levels of safety for yielding and fracture modes, the effect of correlation between $F_y$ and $F_u$ is negligible. This fact is particularly true when the practical target reliability index for yielding and fracture is taken equal to 3.0 and 4.5, respectively. Load models applied in their study were based on the latest data at that time gathered by Ellinwood et al. [5]. Resistance models and correlation concern were characterized in a different study by Najem-Clarke [6].

Schmidt and Bartlett [7] collected statistical data for tension and compression members for four most popular sections. Collected date regarding geometry and material strength for wide flange (W), welded wide flange (WWF), and hollow structural sections (HSS-class C and H) declared slight changes in resistance parameters compare to previous data from 1980s. In some cases new test results disclosed higher coefficient of variation for resistance of steel tension members. Considerable quantity of new collected data was based on experimental evaluation of steel sections produced in 1999 and 2000 by major suppliers to the USA and Canadian markets. In a companion paper, Schmidt and Bartlett [8] utilized mentioned data to re-calibrate the resistance factors in the 1995 National Building Code of Canada. Based on available experimental data, most resistance parameters including geometry, material and discretization factors were proposed in their study. However, professional factors for resistance of axially loaded steel members were chosen from values reported by Chernenko and Kennedy [9] and Kennedy and Gad Aly [10].

The objective of this study is re-calibrating steel tension and compression members design criteria in current AASHTO LRFD bridge design code based on the latest applicable load and resistance models. As the fundamentals of reliability evaluation, approaching a uniform reliability close to target level was pursued in this study. Applied load and resistance models and reliability analysis results are presented in following sections. Finally, suggested modifications based on analysis results are discussed thoroughly.

## 2. Load models

Most important applying loads on highway bridges are dead load, live load (including dynamic effect), wind, earthquake, temperature, etc. In most cases, a combination of dead and live load governs design of a bridge deck system. Clearly, each load component should be considered as a random variable due to the uncertainty in the actual amount of each load.

In this study, latest load models based on existing statistical data are used. A summary of collected data and observations is provided in Calibration of LRFD Bridge Design Code—NCHRP 368 [2]. It should be noted that current load factors in AASHTO Bridge Design Code, are based on a comprehensive reliability study for design of girder-type bridges as the most common bridge system. Hence, it is preferred to use these load factors for all types of bridges to keep an acceptable simplicity in design code. Table 1 shows two load combinations offered for maximum dead and live loads.

Strength I limit state presents basic load combination related to the normal vehicular use of the bridge, while Strength IV limit state is applicable for very high dead load to live load ratios ($r>7$). Values of $r$ may represent the span length in bridge structures in such a way that higher and lower $r$ values stand for longer and shorter spans, respectively. Defining $r'$ as dead load to total load ratio (Eq. (1)), the Strength I limit state is applicable for $r' \leq 0.875$ and Strength IV limit state should be taken for $r'>0.875$. In fact, a practical range of $r'$ values (0.2–0.8) covers most bridges. Consequently, in calculation of reliability indices, the main focus should be on this range.

$$r' = \frac{\mathrm{DL}+\mathrm{DW}}{\mathrm{DL}+\mathrm{DW}+\mathrm{LL}+\mathrm{IM}} \tag{1}$$

According to existing statistical data [2], most suitable distribution functions and their related random parameters have been taken for each load component (Table 2).

Based on cumulative distribution functions for recorded dynamic load factors, IM, for through trusses, deck trusses, and rigid steel frames, the average Coefficient of Variation (COV) is considerably larger than calculated COV for steel or concrete girders ($V=1.125$ vs. $V=0.71$ for steel girders and $V=0.56$ for P/C AASHTO concrete girders). The high ratio of COV can affect the reliability indices for steel tension and compression members which will be discussed further in following sections.

**Table 1**
Load combinations and load factors (AASHTO 2007).

| Limit state | DC | DW | LL | IM |
|---|---|---|---|---|
| Strength I | 1.25 | 1.50 | 1.75 | 1.75 |
| Strength IV | 1.50 | 1.50 | – | – |

DC: components dead load, DW: wearing surface dead load, LL: vehicular live load, and IM: vehicular dynamic load allowance.