# Calibration of Service Limit States for Concrete in AASHTO LRFD Bridge Design Specifications

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ABSTRACT: The strength, or ultimate, limit states (ULS) of the AASHTO LRFD were calibrated through structural-reliability theory to achieve a certain level of safety. In theory, exceeding the strength limit state results in a collapse or failure of a component.

Unlike strength limit states, the consequences of exceeding the service limit states are not well defined. In the past, the service limit states were not statistically-calibrated. This paper presents the work performed to statistically calibrate the service limit states for concrete.

## INTRODUCTION

The notion of limit state is fundamental in the *AASHTO LRFD Bridge Design Specifications* (AASHTO LRFD) (AASHTO 2014). A limit state is defined as the boundary between acceptable and unacceptable performance of the structure or its component.

The strength, or ultimate, limit states (ULS) of the AASHTO LRFD were calibrated through structural-reliability theory to achieve a certain level of safety. Theoretically, exceeding the strength limit state results in a collapse or failure; an event that should not occur any time during the lifetime of the structure. Therefore, there is a need for an adequate safety margin expressed in the form of a target reliability index, $\beta_T$. For bridge girders, the target reliability is taken as, $\beta_T = 3.5$ (Nowak 1999; Kulicki et al., 2007). The strength limit states do not consider the integration of the daily, seasonal, and long-term service stresses that directly affect long-term bridge performance and subsequent service life.

The current service limit states (SLS) of the AASHTO LRFD are intended to ensure a serviceable bridge for the design life; assumed to be 75 years in AASHTO LRFD. When the SLS is exceeded, repair or replacement of components may be needed, and repeatedly exceeding SLS can lead to deterioration and eventually collapse or failure (ULS). In general, SLS can be exceeded, but the frequency and magnitude have to be within acceptable limits.

Originally, the service limit states in AASHTO LRFD were based upon the traditional serviceability provisions of the *Standard Specifications for Highway Bridges* (AASHTO 2002). They were formulated to achieve component proportions similar to those of the *Standard Specifications*. However, these service limit states were not calibrated using reliability theory to truly achieve uniform probability of exceedance as the tools and data necessary to accomplish this calibration were not available to the code writers when AASHTO LRFD was developed.

Even with the development of additional information after the original development of AASHTO LRFD, the development of calibrated service limit states remains a difficult task. The main source of difficulty is the lack of information on the relationship between the frequency of exceeding a certain service limit state and the deterioration of the structure. The lack of this information does not allow the code developers to select the level of reliability required to achieve a level of performance that corresponds to a certain service life.

The pioneering work performed under the Strategic Highway Research Program 2 (SHRP2), Project R19B, and the National Cooperative Highway Research Program (NCHRP) Project 12-83, developed a process for the calibration of the service limit states in AASHTO LRFD. This paper details the challenges in calibrating the service limit states and the process proposed to calibrate the service limit states for concrete structures in the AASHTO LRFD bridge design specifications.

## SELECTION OF LIMIT STATES TO BE CALIBRATED

Service limit states in the then-current AASHTO-LRFD were reviewed. It was determined that not all limit states can be calibrated using available information. The limit states were first divided to "non-load-driven" and "load-driven" limit states. Difference between the two groups is basically in the degree of involvement of externally-applied load components in the formulation of the limit state function. In the non-load-driven SLS, the damage occurs due to deterioration or degradation as a function of time and aggressive environment or as inherent behavior due to certain material properties. Examples of non-load driven SLS include penetration of chlorides leading to corrosion of reinforcement, leaking joints leading to corrosion of the structural components under the joints, and cracking of concrete components due to shrinkage of the concrete and due to changes in temperature. In these examples, the external load occurrence plays a secondary role. The research team determined that the available information is not sufficient to perform a meaningful calibration of the non-load-driven limit states.

On the other hand, in load-driven limit states, the damage occurs due to accumulated applications of external loads, usually live load (trucks). Examples of load-driven limit states include: decompression and cracking of prestressed concrete, cracking of reinforced concrete under applied loads, and, fatigue of concrete and reinforcement under repeated application of live load. The information available on these limit states in the literature, supplemented by additional information developed in the research, was deemed to be sufficient to perform the calibration

## SELECTION OF THE RELIABILITY INDEX

Due to the lack of correlation between the frequency of exceeding a certain service limit state and the deterioration of the structure, the inherent reliability of structures designed to past specifications was used in determining the target reliability level for the calibration. As the inherent level of reliability in existing structures varied from one limit state to another, the target reliability also varied. This is a fundamental difference between the calibration of the strength limit state, where the same level of reliability was used for all limit states, and the calibration of the service limit states. For each service limit state, the ultimate goal of the calibration became to calibrate the limit state to obtain uniform reliability level for the full range of applications and this reliability level is similar to the average reliability inherent in existing structure.

## BASIC STEPS OF THE CALIBRATION PROCESS

Regardless of the level of probabilistic design used to perform LRFD calibration, the steps needed to conduct a calibration are as follows:

- Develop the limit state equation to be evaluated, so that the correct random variables are considered. Each limit state equation must be developed based on a prescribed failure mechanism. The limit state equation should include all the parameters that describe the failure mechanism and that would normally be used to carry out a deterministic design of the structure or structural component.
- Statistically characterize the data upon which the calibration is based (i.e., the data that statistically represent each random variable in

the limit state equation being calibrated). Key parameters include the mean, standard deviation, and coefficient of variation (COV) as well as the type of distribution that best fits the data (i.e. often normal or lognormal).

- Select a target reliability value based on the margin of safety implied in current designs, considering the need for consistency with reliability values used in the development of other *AASHTO LRFD* specifications, the consequence of exceeding the limit state, cost and the levels of reliability for design as reported in the literature for similar structures. If the performance of existing structures that were designed using the current code provisions is acceptable, then there is no need to increase safety margin in the newly developed code. Furthermore, the acceptable safety level can be taken as corresponding to the lower tail of distribution of the reliability indices.
- Determine load and resistance factors using reliability theory consistent with the selected target reliability.

Expanding on the four basic steps outlined above, the framework for calibration of SLS using reliability indices is summarized as follows:

<u>Step 1: Formulate the Limit State Function and Identify Basic Variables</u>. Identify the load and resistance parameters and formulate the limit state function. For each considered limit state, the acceptability criteria were established. In most cases, it was not possible to select a deterministic boundary between what is acceptable and unacceptable. Some of the code-specified limit state functions do not have a physical meaning (e.g. allowable compression stress in concrete).

<u>Step 2: Identify and Select Representative Structural Types and Design Cases</u>. Select the representative components and structures to be considered in the development of code provisions for the SLS.

<u>Step 3: Determine Load and Resistance Parameters for the Selected Design Cases</u>. Identify the design parameters based on typical structural types, loads, and locations (climate, exposure to harsh environment). For each considered element and structure, values of typical load components must be determined.

<u>Step 4: Develop Statistical Models for Load and Resistance</u>. Gather statistical information about the performance of the considered types and models, in selected representative locations and traffic. Gather statistical information about quality of workmanship. Ideally, for given location, and traffic, the required data includes: general assessment of performance, assumed time to initiation of deterioration, assumed deterioration rate as a function of time, maintenance, and repair (frequency and extent). Develop statistical load and resistance models (as a minimum, determine the bias factors and coefficients of variation). The parameters of load and resistance are determined not only by magnitude, as is the case with strength limit states, but also frequency of occurrence (e.g. crack opening) and as a function of time (e.g. corrosion rate, chloride penetration rate). The available statistical parameters were utilized. However, the database is rather limited, and for some serviceability limit states, there is a need to assess, develop, and/or derive the statistical parameters.

The parameters of time-varying loads were determined for various time periods. The analyses were performed for various traffic parameters (average daily truck traffic (ADTT), legal loads, multiple presence, traffic patterns). The load frequencies serve as a basis for determination of acceptability criteria.

<u>Step 5: Develop the Reliability Analysis Procedure</u>. The reliability index for each case can be calculated using closed-form formulas available for particular types of probability distribution functions (PDF) in the literature or Monte Carlo method. In this study, all of the reliability calculations were based on Monte Carlo analysis. The Monte Carlo method is a stochastic technique that is based on the use of random numbers and probability statistics to simulate a large number of computer-based experiments. The outcome of the simulation is a large number of solutions that takes into account all of the random variables in the resistance equation.

<u>Step 6: Calculate the Reliability Indices for Designs Performed Using Current Design Code and</u>

Current Practice. Calculate the reliability indices for selected representative bridge components corresponding to current design and practice.

Step 7: Review the Results and Select the Target Reliability Index, $\beta_T$. Based on the calculated reliability indices, select the target reliability index, $\beta_T$. Select the acceptability criteria, i.e., performance parameters, that are acceptable, and performance parameters that are not acceptable.

Step 8: Select Potential Load and Resistance Factors. Prepare a recommended set of load and resistance factors. The objective is that the design parameters (load and resistance factors) have to meet the acceptability criteria for the considered design situations (location and traffic). The design parameters should provide reliability that is consistent, uniform, and conceivably close to the target level.

Step 9: Calculate Reliability Indices. Calculate the reliability indices corresponding to the recommended set of load and resistance factors for verification. If the design parameters do not provide consistent reliability, modify the parameters and repeat Step 8.

The annual probability of exceedance was used in the statistical analysis of all limit states except for the calibration of the fatigue limit state which was performed based on infinite fatigue life.

For each limit state considered, the calibration process was revised to best fit the limit state. The live load used in this study was based on a large study of weigh-in-motion (WIM) data. The dead load, live load and materials resistance statistics may be found in Wassef et.al. (2014).

## CALIBRATION OF THE LIMIT STATE FOR TENSION IN PRESTRESSED CONCRETE BEAMS, SERVICE III LIMIT STATE

The design of prestressed beams is typically controlled by the tensile stress limits for the concrete under Service III limit state. As such, the calibration for prestressed concrete superstructures was performed for Service III limit state.

An acceptable design will result in maximum tensile stress below the limit set in the specifications when the stress is calculated based on uncracked section under the full design live load for Service III load combination. For typical precast, pretensioned beams, the tensile stress limits, $f_t$, are $f_t = 0.0948\sqrt{f_c'}$ and $f_t = 0.19\sqrt{f_c'}$ for bridges in severe corrosion conditions and in no worse than moderate corrosion conditions, respectively. Even though these limits are below the modulus of rupture for concrete ( $f_t = 0.24\sqrt{f_c'}$ for normal density concrete), this does not mean that cracks will not open under the Service III design load. This can be explained as follows: When the girder is subjected to heavy loads, such as heavy permit or illegal overweight vehicles, the tensile stress in the concrete may exceed the modulus of rupture causing the beam to crack. Subsequent to the formation of the cracks, every time the load on the bridge results in tensile stress in the concrete, i.e. results in decompression, the crack will open. The width of the crack opening depends on the difference between the decompression moment and the actual moment applied. After the load passes, the crack closes again.

Opening of the cracks allows a path for contaminants to reach the prestressing steel. It is expected that the vulnerability of prestressing strands to corrosion increases with the increase of crack width and with the increase in the frequency of crack opening.

Basis of the load factor in AASHTO LRFD: During the early stages of the development of the *AASHTO LRFD Specifications* in the early 1990s, only Service I load combination was considered for calculating all stresses in prestressed concrete components. The load factor for live load was 1.0 which is the same load factor used for service loads under the *AASHTO Standard Specifications*; the predecessor to the *AASHTO LRFD Specifications*.

The design live load specified in the *AASHTO LRFD Specifications* produces higher unfactored, undistributed load effects than that specified in the *AASHTO Standard Specifications*. The girder distribution factors, particularly for interior girders, for many typical girder systems in the *AASHTO LRFD Specifications* are lower than those in the *Standard Specifications* thus reducing the difference between the unfactored distributed load effects in the two specifications. Even with the smaller distribution factor, the unfactored distributed load effects from the *AASHTO LRFD Specifications* were higher for most girder systems. Using the same load factor for the service limit state (1.0) resulted in higher design factored load effects for the *AASHTO LRFD* designs than for those designed to the *AASHTO Standard Specifications*. The results from the trial designs conducted during the development of the *AASHTO LRFD Specifications* indicated the need for a larger number of strands than required by the *AASHTO Standard Specifications*. This would suggest that designs performed under the *AASHTO Standard Specifications* resulted in under-designed components that should have shown signs of cracking. In the absence of widespread cracking, the load factor for live load was decreased to 0.8 and the Service III load combination was created and was specified for tension in prestressed concrete components. This resulted in a similar number of strands for the designs conducted using both *AASHTO Standard* and *AASHTO LRFD Specifications*.

<u>Method of Calculating Prestressing Losses:</u> The *AASHTO LRFD Specifications* (2014) includes three methods for determining the time-dependent prestressing losses. These three methods are:

- Approximate method: Currently, this method is termed: "Approximate Estimate of Time-Dependent Losses" and is the least-detailed. It requires limited calculations to estimate the time-dependent losses. Prior to 2005, the specifications included a simpler approximate method which was termed: "Approximate Lump Sum Estimate of Time-Dependent Losses". The lump-sum method allowed selecting a value for the time-dependent losses from a table. The value varied based on the type of girders and the type and grade of prestressing steel. Some concrete compressive strength requirements were required to be allowed to use this method.

- Time-Step method: This method is highly detailed and is based on tracking the changes in the material properties with time. The loss calculations are based on the time of the application of loads and the material properties at the time of the load application. This method is required to be used in the design of post-tensioned segmental bridges. It may also be used for other types of bridges; however, due to the level of effort required, it is typically limited to segmental bridges.

- Refined Estimates of Time-Dependent Losses: This method is more detailed than the approximate method but less detailed than the time step method. This method is the most used among all three methods.

  Originally, the method of calculating prestressing force losses in *AASHTO LRFD Specifications* (the "pre-2005" method) was the same method used in *AASHTO Standard Specifications*. A new method of loss calculations (the "post-2005" method) first appeared in the 2005 Interim to the Third Edition of *AASHTO LRFD Specifications*. The post-2005 method is thought to produce a more accurate estimate of the losses. The post-2005 method has new equations for calculating the time-dependent prestressing losses and it also introduced the concept of "elastic gain." After the initial prestressing loss at transfer, when load components that produce tensile stresses in the concrete at the strand locations are applied to the girder, the strands are subjected to an additional tensile strain equal to the strain in the surrounding concrete due to the application of the loads. This results in an increase in the force in the strands. The increase in the force in the strands was termed "elastic gain" and the post-2005 prestressing loss method allows including the elastic gain to be used to offset some of the losses.

  When the "elastic gain" was considered, the post-2005 prestressing loss method produced lower prestressing force losses than the earlier method. The reduction in prestressing losses resulted in fewer strands than what was required under the *AASHTO Standard Specifications* and under earlier editions of

*AASHTO LRFD Specifications*. This raised some concern as some practitioners and researchers thought that the higher prestressing losses calculated using the pre-2005 loss method compensated for the lower live load effects caused by the lower design live load used in the *AASHTO Standard Specifications* or the lower load factor used for Service III load combination of *AASHTO LRFD Specifications*. The effect of the prestressing loss method on the design and on the reliability index of prestressed beams was investigated.

Live Load Model

Live load used in the design of study bridges: Traditionally, prestressed concrete components are designed for the number of traffic lanes, including multiple presence factors, that produces the highest load effects. This was assumed to continue in the future and all beams used in the calibration were designed utilizing this approach.

Live load used in determining the reliability indices: As the limit state function, or the physical phenomena, is related to the crack opening, the load used in the calibration had to reflect the load bridges are expected to be subjected to on regular basis; not the maximum design load. The study of WIM data indicated that the presence of heavy trucks simultaneously in adjacent traffic lanes simultaneously is not likely. As such, the load side of the limit state function in the reliability analysis was calculated assuming the live load existed in only one lane and no multiple presence factor was included. The design truck, tandem, and uniform lane load specified in the *AASHTO LRFD Specifications* were used unless otherwise noted. The live load distribution factors specified in the *AASHTO LRFD Specifications* were used in distributing the design loads. The dynamic load allowance used in the original calibration of the strength limit state in *AASHTO LRFD* (10%) was applied to the load side.

The return period considered in the calibration of the Service III limit state was one year. This return period was selected due to the fact that the live load statistics were developed based on 1 year of reliable WIM data from various WIM sites. Furthermore, an ADTT of 5000 was used for the bulk of the calibration. This ADTT is higher than that for the majority of the WIM sites used in the study and, as such, represents a conservative ADTT for most sites.

Methods of Analysis for Study Bridges

Study bridges were analyzed twice. Except for the method used in determining the prestressing losses, both analyses were performed using AASHTO LRFD. The prestressing losses were determined as follows:

- For the first analysis, the prestressing losses were determined using the method termed "Refined Estimates of Time-Dependent Losses" in current AASHTO LRFD and the "elastic gain" was considered.
- In the second analysis, the bridges were analyzed using the method termed "Refined Estimates of Time-Dependent Losses," in AASHTO LRFD prior to 2005. In this case, the "elastic gain" was not allowed

Performing the analysis twice allowed the investigation of the effect of the prestressing loss method on the required number of strands and on the reliability index.

Limit State Functions Investigated

The following three different limit state functions were investigated:

Decompression Limit State: This limit state assumes that the "failure" occurs when the stress in the concrete on the tension face calculated based on the uncracked section under the combined effect of factored dead load and live load ceases to be compression.

- Stress Limit State: This limit state assumes that the "failure" occurs when the tensile stress in the concrete on the tension face under the combined effect of factored dead load and live load exceeds a certain tensile stress limit calculated based on the uncracked section properties regardless of whether the section has been previously cracked or not. Stress limits of $f_t = 0.0948\sqrt{f_c'}$, $f_t = 0.19\sqrt{f_c'}$ and $f_t = 0.25\sqrt{f_c'}$ were initially considered in the reliability analysis, however, a stress limit of $f_t = 0.19\sqrt{f_c'}$ was used for the final calibration.

- Crack Width Limit State: This limit state assumes that the "failure" occurs when the previously formed crack in the concrete opens and the crack width reaches a certain pre-specified crack width. Crack widths of 0.008, 0.012, and 0.016 inches were initially considered in the reliability analysis, however, none produced uniform reliability. The bulk of the calibration was performed using a crack width of 0.016 inches. The differentiation between different environments is accounted for in the calibration through the use of different reliability indices in association with the same crack width.

For each girder, the design was performed based on certain stress limits as is conventionally done and the girder section and number of strands were determined. The reliability index was determined for each of the three limit state functions described above using the same girder design, i.e. the same girder section and same number of strands.

Each of the limit state functions requires a different level of loading before the criteria is violated. As such, the frequency at which any of the three limit states is violated and the corresponding reliability index depend on the level of loading required to cause the limit state to be violated. For a specific cross section with a specific prestressing steel area and force, reaching the decompression limit state requires less applied load than reaching a specified tensile stress which in turn requires less load than that required to reach a specific wider crack width. For any limit state function, requiring higher load to violate a specific limit state means that the section resistance is higher and this results in higher reliability index calculated.

Database of Existing Bridges: To determine the inherent reliability index of existing bridges, a database of existing prestressed concrete girder bridges was extracted from the database of bridges used in the NCHRP 12-78 project (Mlynarski, et al. 2011). The database used in this study included 30 I- and bulb-T girder bridges, 31 adjacent box girder bridges, and 36 spread box girder bridges. The reliability index was calculated for the girders of these bridges for the three limit state functions listed above.

The calculated reliability indices were used to establish the target reliability index for the calibration.

Database of Simulated Bridges A database of simulated simple span bridges was designed using AASHTO I-girder sections for four different cases. The simulated bridges have span lengths of 30, 60, 80, 100, and 140 ft and girder spacing of 6, 8, 10, and 12 ft. This database was analyzed to determine the effect of the change in the method of estimating prestressing losses (pre-2005 and post-2005 methods) and the design environment ("severe corrosive conditions" and "normal" or "not worse than moderate corrosion conditions"). The two environmental conditions are signified by the maximum concrete tensile stress limit ( $f_t = 0.0948\sqrt{f_c'}$ or $f_t = 0.19\sqrt{f_c'}$ ) used in the design. The four cases of design considered were:

Case 1: *AASHTO LRFD* with maximum concrete tensile stress of $f_t = 0.0948\sqrt{f_c'}$ and pre-2005 prestress loss method

Case 2: *AASHTO LRFD* with maximum concrete tensile stress of $f_t = 0.0948\sqrt{f_c'}$ and post-2005 prestress loss method

Case 3: *AASHTO LRFD* with maximum concrete tensile stress of $f_t = 0.19\sqrt{f_c'}$ and pre-2005 prestress loss method

Case 4: *AASHTO LRFD* with maximum concrete tensile stress of $f_t = 0.19\sqrt{f_c'}$ and post-2005 prestress loss method

The average reliability indices for the existing bridges and the simulated bridges for different assumptions of design and for different limit state functions are shown in Table 1.

Calibration Results:
The reliability indices for different limit state functions for bridges designed in accordance with the then-current AASHTO LRFD (2012) were calculated and plotted. The graphs for the decompression limit states are shown below for bridges designed for tensile stress limit in concrete

of $f_t = 0.0948\sqrt{f'_c}$ and $f_t = 0.19\sqrt{f'_c}$ . The graphs for other limit state functions and other stress limits may be found in Wassef et.al. (2014).
Figures 1 and 2 show the results when the girders were designed for a load factor for Service III limit state of 0.8 as was specified in AASHTO LRFD (2012).

As shown in Figures 1 and 2, the reliability index is not uniform for the full range of span length. In addition, the reliability index is generally lower than the inherent reliability of bridges designed based on the prestressing loss method used before 2005. With the majority of bridges designed using the pre-2005 loss method, the inheren reliability of the system is that of the bridges designed to pre-2005 loss method. Therefore, the simulated bridges were redesigned using a load factor of 1.0 instead of the 0.8 existed in the AASHTO LRFD (2012). Figures 3 and 4 show the reliability index for the redesigned bridges.
Figures 3 and 4 show that the designs using a load factor of 1.0 for live load in Service III limit state exhibit more uniform reliability level across the full range of span lengths included in the study. The average reliability indices also closer to the target reliability indices of 1.2 and 1.0 for bridges designed for concrete tensile stress limits of $0.0948\sqrt{f'_c}$ and $0.19\sqrt{f'_c}$ , respectively.

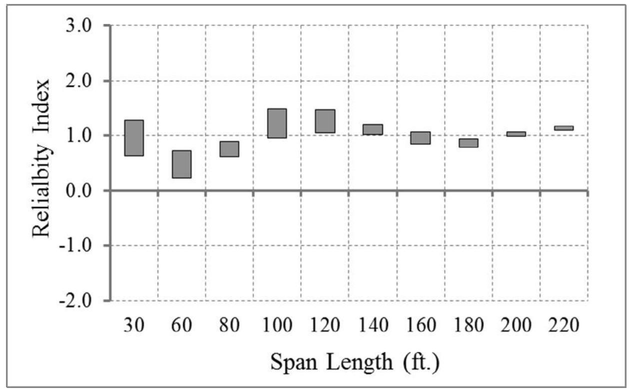

Figure 1. Reliability indices for bridges at decompression limit state (ADTT=5000), $\gamma_{LL}$=0.8, ( $f_t = 0.0948\sqrt{f'_c}$ )

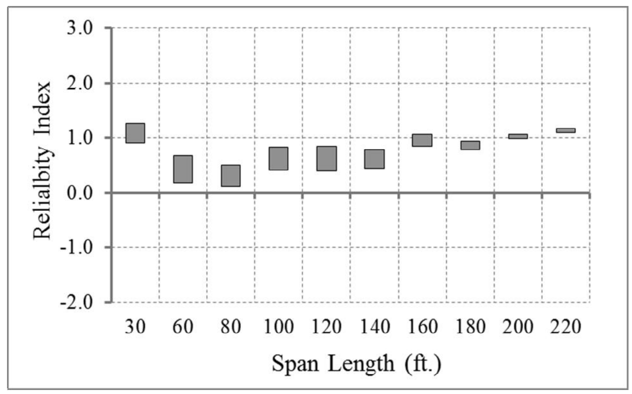

Figure 2. Reliability indices for bridges at decompression limit state (ADTT=5000), $\gamma_{LL}$=0.8 ( $f_t = 0.19\sqrt{f'_c}$ ).

Table 1. Reliability Indices for Existing and Simulated Bridges (Return Period of 1 Year and ADTT 5000)

| Performance Level | Reliability Index | | | | |
|---|---|---|---|---|---|
| | Average β for Existing Bridges in the NCHRP 12-78 | Average β for Simulated bridges designed for $f_t = 0.0948\sqrt{f'_c}$ and pre-2005 loss method | Average β for Simulated bridges designed for $f_t = 0.19\sqrt{f'_c}$ and pre-2005 loss method | Proposed Target β for bridges in severe environment | Proposed Target β for bridges in normal environment |
| Decompression | 0.74 | 1.44 | 1.07 | 1.20 | 1.00 |
| Maximum Allowable Tensile Stress of $f_t = 0.19\sqrt{f'_c}$ | 1.05 | 1.80 | 1.43 | 1.50 | 1.25 |
| Max. Allowable Crack Width of 0.016 in. | 2.69 | 3.68 | 3.15 | 3.30 | 3.10 |

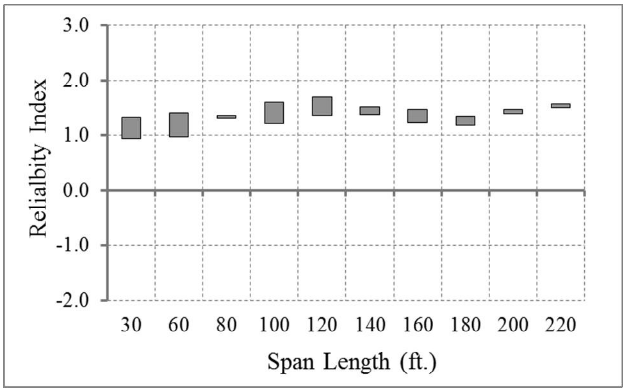


Figure 3. Reliability indices for bridges at decompression limit state (ADTT=5000), $\gamma_{LL}$=1.0 ( $f_t = 0.0948\sqrt{f_c'}$ ).

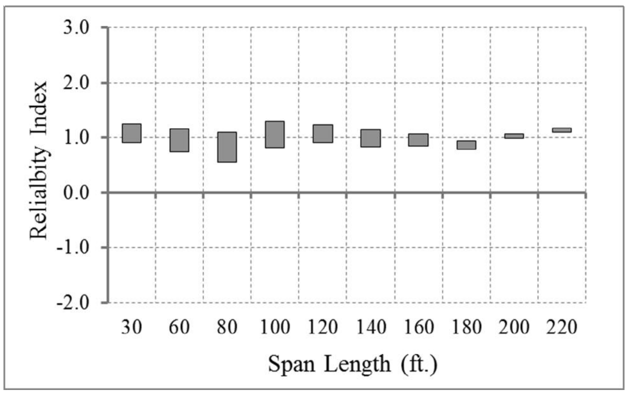


Figure 4. Reliability indices for bridges at decompression limit state (ADTT=5000), $\gamma_{LL}$=1.0 ( $f_t = 0.19\sqrt{f_c'}$ ).

Based on the study, it was proposed that the load factor for live load in the Service III limit state be increased from 0.8 to 1.0. AASHTO accepted the recommendation and it was incorporated in the AASHTO LRFD Design Specifications.

## CALIBRATION OF THE LIMIT STATE FOR CONTROLE OF CRACKING IN REINFORCED CONCRETE, SERVICE I LIMIT STATE

The distribution of reinforcement is used to control of reinforced concrete in AASHTO LRFD. Tighter spacing of reinforcement results in larger numbers of narrow cracks. The narrow cracks result in better resistance to contaminant penetration thus reducing the reinforcement exposure to corrosive agents.

Two exposure classifications exist in *AASHTO LRFD*: Class 1 exposure condition and Class 2 exposure condition. Class 1 relates to an estimated maximum crack width of 0.017 in. while Class 2 relates to an estimated maximum crack width of 0.01275 in. Class 2 is typically used for situations where the concrete is subjected to severe corrosion conditions such as bridge decks exposed to deicing salts and substructures exposed to water. Class 1 is used for less corrosive conditions and could be thought of as an upper bound in regards to crack width for appearance and corrosion. Previous research indicates that there appears to be little or no correlation between crack width and corrosion. However, the different classes of exposure conditions have been so defined in the design specifications in order to provide flexibility in the application of these provisions to meet the needs of the bridge owner.

The available information allowed the calibration of decks. Other types of components were not included in the calibration.

Reinforced concrete decks designed using the conventional method are designed for the heavy axles of the design truck. This required developing the statistical parameters of the axle loads of the trucks in the WIM data. (Wassef et. al. 2014). Statistical parameters corresponding to a one year return period were assumed in the reliability analysis. ADTTs of 1000, 2500, 5000, and 10,000 were considered, however, an ADTT of 5000 was used as the basis for the calibration.

Due to the lack of clear consequences for violating the limiting crack width, there was no basis for changing the nature or the limiting values of the limit state function, i.e. the crack width criteria. The work was based on maintaining the current crack width values and calibrating the limit state to produce a uniform reliability index similar to the average reliability index produced by the current designs.

A database of decks representing commonly used proportions was developed and used in the calibration. The characteristics of the decks are shown in Table 2.

Table 2. Summary Information of 15 Bridge Decks Designed using *AASHTO LRFD* Conventional Deck Design Method

| Deck Group # | Girder Spacing (ft.) | Deck Thickness (in.) |
|---|---|---|
| 1 | 6 | 7.0 |
| | | 7.5 |
| | | 8.0 |
| 2 | 8 | 7.5 |
| | | 8.0 |
| | | 8.5 |
| 3 | 10 | 8.0 |
| | | 8.5 |
| | | 9.0 |
| | | 9.5 |
| 4 | 12 | 8.0 |
| | | 8.5 |
| | | 9.0 |
| | | 9.5 |
| | | 10.0 |

The reliability indices are dependent on the ADTT and they decrease as the ADTT increases. The case of ADTT of 5000 was used as the base case. For this ADTT, the reliability index inherent in current designs for the negative moment region and 1-year return period was calculated as 1.61 and 1.05 for Class 1 and Class 2, respectively. Based on these values, target reliability indices of 1.6 and 1.0 were selected for Class 1 and Class 2, respectively.

Figures 5 through 8 show the reliability indices determined for the deck in the database. In all cases, the load factor for live load for the Service I limit state was taken as 1.0.

The results showed that the variation in the reliability indices is relatively small and that current designs give uniform reliability indices for the range of girder spacing considered. As such, it was concluded that current design provisions do not need to be revised.


Figure 5. Reliability Indices of Various Bridge Decks Over A 1 Year Return Period (ADTT=5000), Positive Moment Region, Class 1 Exposure


Figure 6. Reliability Indices Of Various Bridge Decks Over A 1 Year Return Period (ADTT=5000), Negative Moment Region, Class 1 Exposure

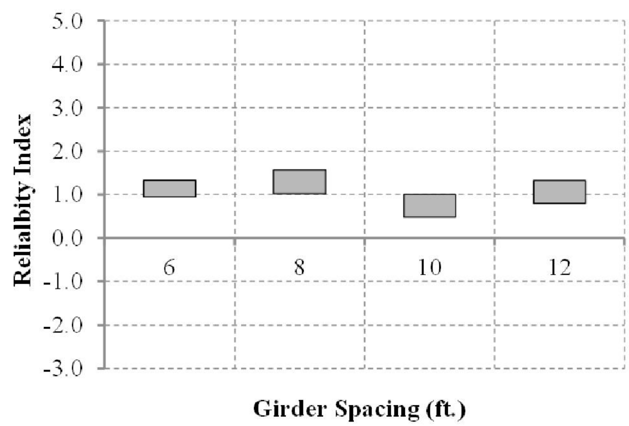


Figure 7. Reliability Indices Of Various Bridge Decks Over A 1 Year Return Period (ADTT=5000), Positive Moment Region, Class 2 Exposure


Figure 8. Reliability Indices Of Various Bridge Decks Over A 1 Year Return Period (ADTT=5000), Negative Moment Region, Class 2 Exposure

## CALIBRATION OF THE FATIGUE LIMIT STATE FOR REINFORCEMENT AND CONCRETE IN COMPRESSION

The limit on the compressive stress in prestressed concrete is actually intended to control fatigue of concrete in compression. Fatigue of reinforcement and concrete in compression are both checked assuming infinite fatigue life. The study of WIM data determined the appropriate load factor for current traffic for both finite and infinite fatigue life (Kulicki et. al. 2015). The process for calibrating the fatigue limit state for structural steel was also developed by Kulicki. et. al. (2015) and was used to calibrate the fatigue limit state for reinforcement and in concrete.

Test results of fatigue in reinforcement and concrete in compression were collected and used in the calibration. The list of tests and the results may be found in Wassef et. al. (2014)

### Fatigue of Steel Reinforcement in Tension in AASHTO LRFD

The infinite fatigue life threshold in AASHTO LRFD for straight reinforcing bars and welded-wire reinforcement without a cross weld in the high-stress region (defined as one-third of the span on each side of the section of maximum moment) is specified as:

$$(\Delta F)_{TH} = 24 - 20 f_{min} / f_y$$

For welded-wire reinforcement with a cross weld in the high-stress region, the fatigue resistance is specified as:

$$(\Delta F)_{TH} = 16 - 0.33 f_{min}$$

Where $f_{min}$ is the minimum stress.

Results from past studies used to define the fatigue resistance of steel reinforcement were reanalyzed to estimate constant-amplitude fatigue thresholds for every case that can be identified in the research to determine their uncertainty, in terms of bias, mean, and coefficient of variation (COV). The various thresholds were grouped together to make design practical.

The calibration resulted in revising the fatigue resistance equations as follows:

For straight reinforcing bars and welded-wire reinforcement without a cross weld in the high-stress region (defined as one-third of the span on each side of the section of maximum moment):

$$(\Delta F)_{TH} = 26 - 22 f_{min} / f_y$$

For welded-wire reinforcement with a cross weld in the high-stress region, the fatigue resistance is specified as:

$$(\Delta F)_{TH} = 18 - 0.36 f_{min}$$

The revised equations results in higher fatigue resistance in all cases for straight reinforcing bars and welded-wire reinforcement without a cross weld in the high-stress region. The revised equations also result in higher fatigue resistance for all practical cases for welded-wire reinforcement with cross welds (all cases with $f_{min} \leq 50$ ksi).

### Concrete in Compression

The compressive stress limit of 0.40 $f_c'$ for fully prestressed components in other than segmentally constructed bridges of *AASHTO LRFD* Article 5.5.3.1 applies to a combination of the live load specified in the Fatigue I limit state load combination plus one-half the sum of the effective prestress and permanent loads after losses, i.e. a load combination derived from a modified Goodman diagram. This suggests that it represents an infinite-life check as the Fatigue I limit state load combination corresponds with infinite fatigue life.

For this study, the research used to define these S-N curves, Hilsdorf and Kesler (1966) was re-evaluated to estimate the constant-amplitude fatigue threshold, the infinite-life fatigue resistance. The uncertainty of the fatigue resistance was quantified in terms of bias, mean, and coefficient of variation.

The reliability index calculated for designs performed using current design practices was 0.9. This value was close to the target reliability index of 1.0. Therefore, no revisions to the current design specifications were recommended.

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REFERENCES

*AASHTO LRFD Bridge Design Specifications*, 6th ed. 2012. AASHTO, Washington, DC.

*AASHTO LRFD Bridge Design Specifications*, 7th ed. 2014. AASHTO, Washington, DC.

*AASHTO Standard Specifications for Highway bridges, 1*7th ed. 2002. AASHTO, Washington, DC.

Hilsdorf, H., and C. Kesler. 1966. Fatigue Strength of Concrete under Varying Flexural Stresses. *Journal Proceedings, American Concrete Institute*, Vol. 63, No. 10, pp. 1059–1076.

Kulicki, J., W. Wassef, D. Mertz, A. Nowak, N. Samtani, and H. Nassif. 2013. *Service Load Design for 100-Year Life*. SHRP 2, R19B. Transportation Research Board of the National Academies, Washington, DC (Under Preparation).

Kulicki, J., Z. Prucz, C. Clancy, D. Mertz, and A. Nowak. 2007. *Updating the Calibration Report for AASHTO LRFD Code*. Report on NCHRP 20-7/186. Transportation Research Board of the National Academies, Washington, DC.

Mlynarski, M., W. Wassef, and A. Nowak. 2011. *NCHRP Report 700: A Comparison of AASHTO Bridge Load Rating Methods*. Transportation Research Board of the National Academies, Washington, DC.

Nowak, A. 1999. *NCHRP Report 368: Calibration of LRFD Bridge Design Code*. TRB, National Research Council, Washington, DC.

Wassef, W. G., J. M. Kulicki, H. A. Nassif, D. R. Mertz, and A. S. Nowak. 2014. Calibration of LRFD Concrete Bridge Design Specifications for Serviceability, NCHRP Web-Only Document 201, Transportation Research Board, National Research Council, Washington, DC.