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Dennis Mertz
Symposium on Design and
Evaluation of Concrete Bridges

Editors:
Andrzej S. Nowak and Hani Nassif

SP-340



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Dennis Mertz Symposium on Design and Evaluation of Concrete Bridges

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PREFACE

Dennis Mertz Symposium on Design and Evaluation of Concrete Bridges

Professor Dennis Mertz passed away after a prolonged battle with cancer. He spent a large portion of his professional career working on advancing of the state-of-the-art of bridge engineering. He was a great friend and colleague to many at ACI and ASCE. Joint ACI-ASCE Committee 343, joined with ACI Committees 342 and 348, sponsored four sessions to honor his contributions and achievements in concrete bridge design and evaluation. These sessions highlighted the important work and collaborative efforts that Dr. Mertz had with others at ACI and ASCE on various topics. These sessions also combined the efforts among ACI and ASCE researchers and practitioners in addressing various topics related to the design and evaluation of concrete bridges. The scope and outcome of the sessions are relevant to ACI's mission. They raise awareness on established design methodologies applied for various limit states covering topics related flexure, shear, fatigue, torsion, etc. They address problems related to emerging design and evaluation approaches and recent development in design practices, code standards, and related applications. The Symposium Publication (SP) is expected to be an important reference in relation to design philosophies and evaluation methods of new and existing concrete bridges and structures.

Editors

Andrzej S. Nowak and Hani Nassif

Associate Editor

Victor Aguilar

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Updating AASHTO Concrete Design Provisions - Prof. Mertz's Final Major Contribution to AASHTO LRFD

John M. Kulicki, Ph.D., P.E., NAE and Gregg A. Freeby, P.E.

Synopsis: Dr. Dennis Mertz was involved with the *AASHTO LRFD Bridge Design Specifications* [1] for 30 years. Starting with the original development of the specifications and continuing with maintenance and related course development and presentations. His last major contribution to the Specifications was to serve as Principal Investigator for the reorganization of Section 5, Concrete Structures. This presentation summarizes the changes to the structure of the Section including the increased emphasis on design of “B” and “D” regions of flexural members and introduces new and expanded material on beam ledges and inverted T-caps, shear and torsion, anchors, strut and tie modeling and durability. The product of this work was included in the 8th Edition of the Specifications as a complete replacement of Section 5.

Keywords: concrete bridge design, STM, disturbed beam regions, concrete anchors, beam ledges, node efficiency factors

ORGANIZATION AND IDENTIFYING EXPECTATIONS

Support for rewriting Section 5 of the *AASHTO LRFD Bridge Design Specifications* [1] (AASHTO LRFD) was provided by a pool fund project with the Kansas DOT as the lead agency with participation from Indiana, Iowa, Louisiana, Michigan, Nebraska, New Jersey, Ohio, Oregon, Pennsylvania, Texas, Utah, Virginia and Washington. Oversight was provided by Technical Committee 10, Concrete Structures, which had members from Kansas, Nebraska, Pennsylvania, New Hampshire, Louisiana, Tennessee, Virginia, Texas, Minnesota, Wisconsin, California, Oregon, Florida, Washington, as well as the FHWA. Industry liaison was provided by the Prestressed Concrete Institute and the American Segmental Bridge Institute. Some interested parties and subject matter experts participated on an as-needed basis.

The Scope consisted of a survey of stake holders to determine the degree of reorganization and revision that was desired, a critical review of the changes made since original adoption in 1993 as embodied in past Interim Specifications as well as anticipated future changes embodied in the T-10 Working Agendas, development of a revised annotated Table of Contents, writing of a new, revised and reorganized Section 5, and finalizing the document for consideration by T-10 and eventually the Subcommittee on Bridges and Structure (now Committee on Bridges and Structures). In the process seven drafts were written and reviewed, six working meetings with the augment T-10 were held, and almost 1,000 comments were received and reviewed.

The survey of stake holders produced important guidance for the project. Specifically:

- it became clear that evolution of the existing provisions was desired as opposed to revolutionary changes,
- the current system of units involving kips would be retained,
- the distinction between bending regions and disturbed regions should further strengthen and emphasized,
- the existing provisions for bending and axial force design would be retained with only clarification and minor editing,
- the number of methods for designing for shear would be reduced by eliminating the relatively recently readopted “Vci-Vcw” method,
- the differences between shear and torsion design in the general provisions and the provisions for segmental concrete bridges would be harmonized to the extent practical and acceptable to all parties, and;
- Reinforcing details, prestressing details, & seismic details would be consolidated into 3 separate articles.

OVERVIEW OF CHANGES

The following summary of changes serves as a high-level roadmap for the revised Section 5:

- The following articles are largely unchanged: Scope except for introduction of B and D Regions, Definitions, Notations, Material Properties with some deletions, Limit states and Design Methodologies except for new material related to B and D Regions;
- Design for Flexure and Axial Force Effects is relocated to Article 5.6 and modified to reflect B and D Regions;
- Shear and Torsion moved to Article 5.7, is noted to apply to B-Regions, “Vci-Vcw” provisions removed, remaining segmental provisions are relocated to Article 12.5, and the principal stress check is moved to Strength Limit State provisions Article 5.9.2.3.3 from Service limit State provisions where it was erroneously located when originally adopted;
- Design of D-Regions, including STM provisions from 2016 interim revisions which were developed under the reorganization project but were pre-approved earlier to facilitate adoption of the reorganization, design of general zones by behind post-tensioning anchors, STM design of brackets and corbels, revised provisions for design of beam ledges, and design of deep beams;
- Although STM is preferred, some legacy methods still permitted for D-regions for deep beams, brackets and corbels, and beam ledges;
- Prestressing provisions have been collected from three previous articles (5.9, 5.10 and 5.11) and provide better separation of provisions for pretensioning and post-tensioning under the sub-articles on general provisions, stress

limitations including those for principal stresses, losses, and further provisions for pretensioning and for post-tensioning;

- Reinforcing provisions have been collected from two previous articles (5.10 and 5.11)
- A new article on anchors has been added which will be discussed further below, and;
- Provisions for durability have been expanded and relocated to Article 5.14 and include
- Design Concepts, Major Chemical and Mechanical Factors, Concrete Cover by cross reference to table in 5.10.1, Protective Coatings, Deck Protection systems, and Protection for Prestressing Tendons.

B AND D REGIONS

As indicated above, the survey of stakeholders revealed a desire to place more emphasis on the recognition of the structural behaviors associated with B- and D-Regions. To do that design provisions were organized as follows:

- 5.5 – Limit States & Design Methodologies
- 5.6 – Design for Flexural & Axial Force Effects – B Regions
- 5.7 – Design for Shear & Torsion – B Regions
- 5.8 – Design of D-regions
 - 5.8.1 – General
 - 5.8.2 – Strut and Tie Method (STM)
 - 5.8.3 – Elastic Stress Analysis
 - 5.8.4 – Approximate Stress Analysis & Design

Accordingly, AASHTO LRFD requires that regions of a concrete structure be characterized by their behavior as either B- (beam or Bernoulli) or D- (disturbed or discontinuity) Regions. Bernoulli's hypothesis of straight-line strain profiles, in other words, conventional beam theory, applies in B-Regions. Design practices for B-Regions are specified to be based on a sectional model for behavior. Design for flexure in B-Regions is based upon the conventional beam theory of Article 5.6 while the design for shear in B-Regions is based on conventional beam theory in conjunction with the truss analogy of Article 5.7. For B-Regions conventional beam theory is applicable to all limit states. Sectional models are appropriate for the design of typical bridge girders, slabs, and other regions of components where the assumptions of traditional engineering beam theory are valid. This theory assumes that the response at a particular section depends only on the calculated values of the sectional force effects, i.e., moment, shear, axial load, and torsion, and does not consider the specific details of how the force effects were introduced into the member.

A more complex variation in stress and strain exists in D-Regions as shown in Figure 1, where the effective depth of the member, d , is defined as the distance between the extreme compression fiber and the centroid of the primary longitudinal reinforcement. D-Regions encompass locations with abrupt changes in geometry or concentrated forces. Based upon St. Venant's principle, it may be assumed that a D-Region occupies a region comprising one member depth on either side of the discontinuity in geometry or force.

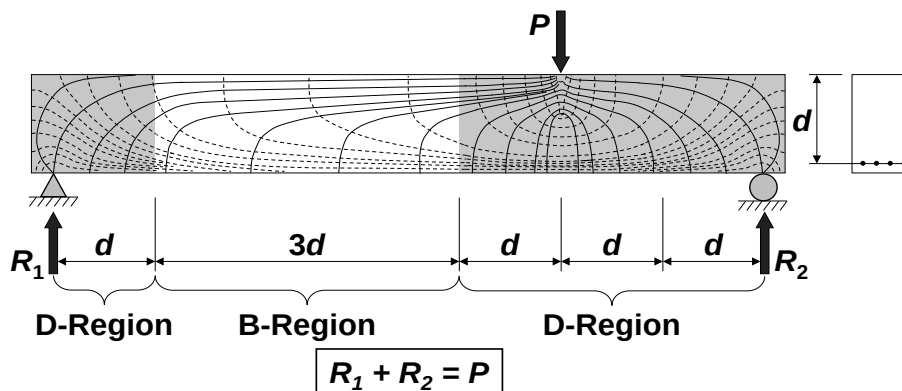


Figure 1—Stress Trajectories within B- and D-Regions of a Flexural Member (adapted from Birrcher [2])

In the commentary to Article 5.5.1.2.3 AASHTO LRFD recognizes three general classes of analysis methods that may be used for the design of D-Regions which are listed as:

- The Strut and Tie Method (STM);
- Elastic Analysis introduced in Article 5.8.3;
- Legacy practices which are empirical and other approximate methods usually developed before refined analysis methods such as the STM became more widely used and are described in Article 5.8.4. Use of these methods is not preferred and is expected to decline in time.

REVISIONS TO STRUT AND TIE PROVISIONS

The decision to include STM provisions in the first edition in 1994 provided designers with a tool that enabled them to accurately and relatively simply analyze situations where the conventional mechanics approach simply did not apply. For example, the traditional method assumes that the shear stress distribution is essentially uniform over the depth and that the longitudinal strains will vary linearly over the depth of the beam. For members such as deep beams, these assumptions are not valid and the behavior can be predicted more accurately if the flow of forces through the complete structure is studied. The STM is applicable to both B- and D-Regions, but it is typically not practical to apply it to B-Regions. Detailed information on this method is given by Schlaich, et al. [3], Collins and Mitchell [4], Martin and Sanders [5], Birrcher, et al. [2]; Mitchell and Collins [6], Williams, et al. [7] and Larson, et al. [8]. The basic steps in the STM usually consist of:

1. Determining the locations of the B- and D-Regions.
2. Defining load cases.
3. Analyzing structural components.
4. Sizing structural components using the shear serviceability check, given by Eq. C5.8.2.2-1.
5. Developing a strut-and-tie model. See Article 5.8.2.2.
6. Proportioning ties.
7. Performing nodal strength checks. See Article 5.8.2.5.
8. Proportioning crack control reinforcement. See Article 5.8.2.6.
9. Providing the necessary anchorage for ties.

In intervening years since first implementation in AASHTO LRFD continued research has led to improvements in the method. It became clear that revisions to the provisions in AASHTO LRFD were needed. To facilitate design of D-Regions using STM, the provisions have been extensively updated based primarily on large scale research conducted at the University of Texas at Austin (Birrcher et al, [2]). Typically, the updates result in less conservative but more realistic estimated of capacity compared to test results, but still provide ample safety as shown in a comparison of these provisions and five other design methods to 179 experimental results in Birrcher, et al. [2].

The revised provisions provide:

- More seamless transition in capacity predictions between D- and B-Regions;
- Improved representation of strut strength at shallow angles;
- Improved determination of the concrete strut area when there are only two stirrup legs,
- and;
- Inclusion of the effect of restraint at reactions and load points which increases the compressive capacity of struts.

The implementation of these improvements is explained below.

The provisions apply to components with reinforcement yield strengths not exceeding 75.0 ksi (520 MPa) and with normal weight concrete compressive strengths for use in design up to 15.0 ksi (105 MPa). Use of the STM for only normal weight concrete components is based solely on a lack of suitable experimental verification.

Plasticity-based methods, which include STM, can be classified into two categories: upper-bound and lower-bound methods. The upper-bound solutions approach the resistance from above, i.e the unconservative side, while lower-

bound solutions approach the resistance from below, i.e. the conservative side. STM is a lower-bound design method. As such, it adheres to the following principles: (1) the truss model is in equilibrium with external forces, and (2) the concrete element has enough deformation capacity to accommodate the assumed distribution of forces (Schlaich et al., [3]). Proper anchorage of the reinforcement is required. Additionally, the compressive forces in the concrete must not exceed the factored strut capacities, and the tensile forces within the strut-and-tie model must not exceed the factored tie capacities. The Commentary to Article 5.8.2.2 provides considerable guidance of establishing an accurate and efficient STM. The angle between the axis of a strut and tie should be limited to angles greater than 25 degrees.

One of the issues with the previous implementation of STM was a discontinuity in shear and flexural reinforcement requirements that was often observed at the interface between the STM based design in a D-Region and a section model design in the adjacent B-Region. The previous design method was found to be increasingly more conservative as the ratio of shear span to depth increased and the associated strut became flatter. The revised STM provisions allow a more seamless transition between B- and D-Regions of a given structure. This is accomplished by replacing the previous strain-based strut efficiency calculations with an efficiency factors based primarily upon the *fib Model Code for Concrete Structures*, (CEB [9]). The efficiency factors have the advantages of being simpler to use and are less open to misinterpretation. They also exhibit better statistical parameters, i.e. bias and coefficients of variation, leading to improved accuracy and precision in a variety of applications as documented by Bircher, et al. [2], Williams, et al. [7] and Larson, et al. [8].

The geometry of each node should be defined prior to conducting the strength checks. Nodes may be proportioned in two ways: as hydrostatic nodes or as nonhydrostatic nodes. Hydrostatic nodes are proportioned in a manner that causes the stresses applied to each face to be equal. Nonhydrostatic nodes, however, are proportioned based on the origin of the applied stress. For example, the faces of a nonhydrostatic node may be sized to match the depth of the equivalent rectangular compression stress block of a flexural member or may be based upon the desired location of the longitudinal reinforcement (see Figure 2). This proportioning technique allows the geometry of the nodes to closely correspond to the actual stress concentrations at the nodal regions. In contrast, the use of hydrostatic nodes can sometimes result in unrealistic nodal geometries and impractical reinforcement layouts as shown in Figure 2. Thus, nonhydrostatic nodes are preferred in design and are used throughout these specifications. The efficiency factors used with Eq 1 apply only to nonhydrostatic nodes. The efficiency factors combined with the increased internal moment arm associated with the use of non-hydrostatic nodes as shown in Figure 2 result in a more accurate representation of the strength of flat struts.

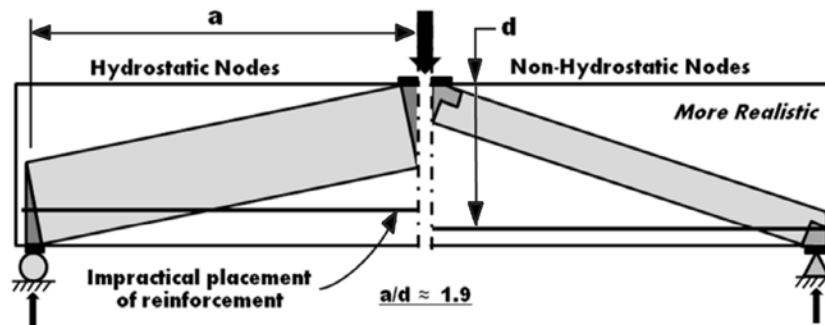


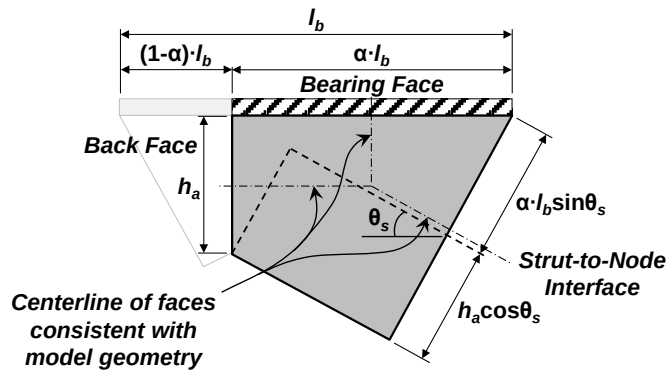
Figure 2—Hydrostatic and Nonhydrostatic Nodes

The previous provisions limited the out-of-plane width of concrete that could be included in strut to six diameters of the enclosed bar on each side a stirrup leg, or the cover if less. If there were only two stirrup legs in a given cross section this limited the effective width of a strut to twelve bar diameters plus two cover dimensions, on the order of about 20 to 22 inches (about 500 mm to 550 mm) in many cases. Based on test results this restriction has been dropped. Using the new provisions, the out-of-plane dimension of the node should be based on the bearing(s) and

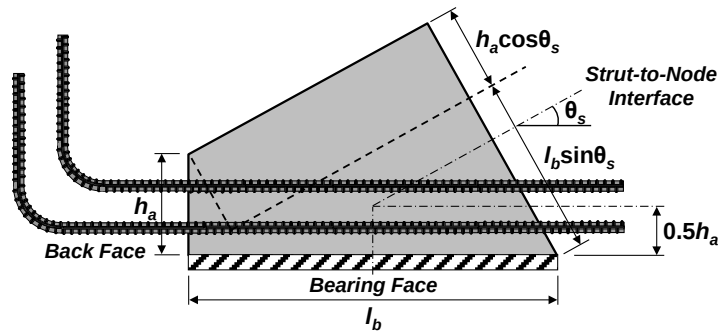
member geometries. The commentary has some additional advice regarding beam widths greater than about 36 inches (about 900mm).

As illustrated in Figure 3, nodes at the intersection of struts and ties may be characterized as CCC: nodes where only struts intersect, CCT: nodes where a tie intersects the node in only one direction, and CTT: nodes where ties intersect in two different directions. The following notation applies:

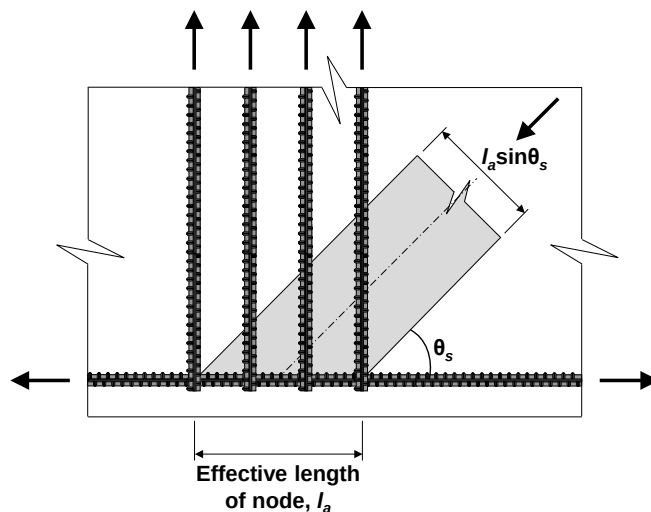
- h_a = length of the back face of a node
- l_a = effective length of a CTT node
- l_b = length of the bearing face
- α = fraction defining the bearing face length of a portion of a nodal region
- θ_s = angle between strut and longitudinal axis of the member (degrees)



(a) CCC Node



(b) CCT Node



(c) CTT Node

Figure 3—Nodal Geometries

Once the forces in the struts and ties are calculated the strength limit state capacities of the struts, ties and nodes are determined. Determination of the tensile capacity of ties is relatively straight forward. ASHTO LRFD provides a compressive stress limit at the node face, f_{cu} , given by Equation 1, applied with consistent units, for the strength limit state unless confinement reinforcement is provided complying with the caveats in the Specification. The determination of the compressive capacity involves the efficiency factor, v , previously noted. It also contains a factor, m , to account for triaxial restraint where appropriate to the geometry of bearings and load points, thus overcoming another drawback of the previous provisions.

$$f_{cu} = mvf'_c \quad (\text{Equation -1})$$

where:

- f'_c = compressive strength of concrete for use in design
- m = confinement modification factor defined in Article 5.6.5
- v = concrete efficiency factor:
 - 0.45, structures that do not contain crack control reinforcement as specified in Article 5.8.2.6;
 - as shown in Table 1 for structures with crack control reinforcement as specified in Article 5.8.2.6
- A_1 = area under the bearing device
- A_2 = notional area specified in Article 5.6.5

Table 1—Efficiency Factors for Nodes with Crack Control Reinforcement (f'_c in consistent units)

Face	Node Type		
	CCC	CCT	CTT
Bearing Face/Back Face	0.85	0.70	
Strut-to-Node Interface	$0.85-f'_c/A$	$0.85-f'_c/A$	$0.85-f'_c/A$
	$A=20 \text{ (KSI)}$ $A=140 \text{ (MPa)}$	$A=20 \text{ (KSI)}$ $A=140 \text{ (MPa)}$	$A=20 \text{ (KSI)}$ $A=140 \text{ (MPa)}$
	$0.45 \leq v \leq 0.65$	$0.45 \leq v \leq 0.65$	$0.45 \leq v \leq 0.65$

The compressive stress limit given by Equation 1 need only be checked at the face of nodes at the ends of the strut, and not within the length of the strut. This criterion is based on experiments that showed cacked and crushed concrete patterns in which the compressive stress trajectories converged at the nodes and diverged away from the nodes. The resulting shape of the struts is often called a “bottle shape strut as shown in Figure 4 (Birrcher [2])

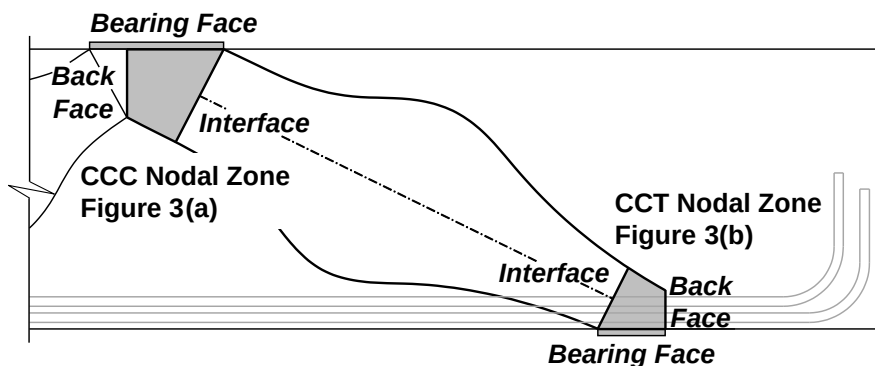


Figure 4— Schematic of bottle-shaped strut

Eq. 1 has been found to be valid for design concrete compressive strengths up to 15.0 ksi (105 MPa) and therefore stress-block factors, k_c and α_1 , for high-strength concrete need not be applied to this equation. The Commentary to Article 5.8.2.5.3a provides graphics of the type of nodes in Table 1 showing the application of the efficiency factors.

In addition to satisfying strength criteria, the node regions are designed to comply with the stress and anchorage limits specified in Articles 5.8.2.4.1 and 5.8.2.4.2. Crack control reinforcement is required by Article 5.6.2. This reinforcement is intended to control the width of cracks and to ensure a minimum ductility for the member so that significant redistribution of internal stresses is possible if required to mobilize the full strength. This behavior is consistent with the plasticity basis of STM.

In summary, the proposed provisions take advantage of results from an extensive research effort sponsored by the Texas DOT at UT-Austin which involved a thorough examination of previous tests, primarily of deep beams, additional large scale deep beam tests at UT-Austin, and a comparison of current provisions and those which have been used in Europe for many years. The most significant changes in the proposed provisions are:

- Elimination of distributed reinforcement if an associated efficiency factor is used;
- Use of simple concrete efficiency factors similar to those in the fib model Code;
- Use of the existing AASHTO confinement factor to increase the usable concrete strength where there is clear distance on all sides of a bearing plate or load plate;
- Provision of expanded design rules to size the nodes in the STM;
- Use of a single panel truss model for shear spans up to 2.0;
- Elimination of principal tensile strain as a criterion for nodal capacity, and;
- Elimination of a separate strut capacity check away from the nodes.

The resulting capacities are in good agreement with test results and have been shown to have a smaller bias than the current provisions, while still providing ample safety, and a significantly smaller Coefficient of Variation (COV) indicating improved precision and consistency.

NEW PROVISIONS FOR PRE- AND POST-INSTALLED ANCHORS

Introduction

The inclusion of Article 13 for anchors is new to AASHTO LRFD. Rather than start from scratch it was decided to take advantage of the well-developed, consensus-based provisions in ACI 318-14 Chapter 17 [10] by linked the new provisions to ACI with some exceptions, most of which are discussed below. In fact, Article 13 states that the ACI Anchor provisions are incorporated by reference unless specially modified or excluded by the provisions of new Article 13. Thus Article 13 is intended to provide guidance on types of anchor covered, design conditions that may need to be investigated, and modifications to ACI, and applications specifics. The concrete compressive strength used for design shall not exceed 10.0 ksi (70 MPa) for cast-in-place anchors or 8.0 ksi (55 MPa) for post-installed anchors; the provisions do not apply to grouted anchors. All design, material, testing, anchor, geometry and depth limitations, anchor spacing, edge distances and, acceptance requirements specified in ACI 318-14 Chapter 17 [10] apply to anchors designed or supplied under the AASHTO LRFD provisions. Group effects, eccentricity of loading, presence or lack of anchor reinforcement, and the possibility of concrete cracking or splitting are to be considered in the design. Lightweight concrete factors for anchor design are to be as specified in ACI 318-14 Chapter 17 [10].

The various types of anchors covered by ACI 318-14 Chapter 17 [10], and therefore by the new AASHTO LRFD provisions are listed below and shown in Figure 5:

- Headed studs and headed bolts;
- Hooked bolts having a geometry that has been demonstrated to result in a pullout strength without the benefit of friction in uncracked concrete equal to or exceeding $1.4N_p$, where N_p is given in ACI 318-14 Eq. 17.4.3.5 [10];
- Post-installed expansion and undercut anchors meeting the assessment criteria of ACI 355.2 [11], and;
- Adhesive anchors meeting the assessment criteria of ACI 355.4 [12].

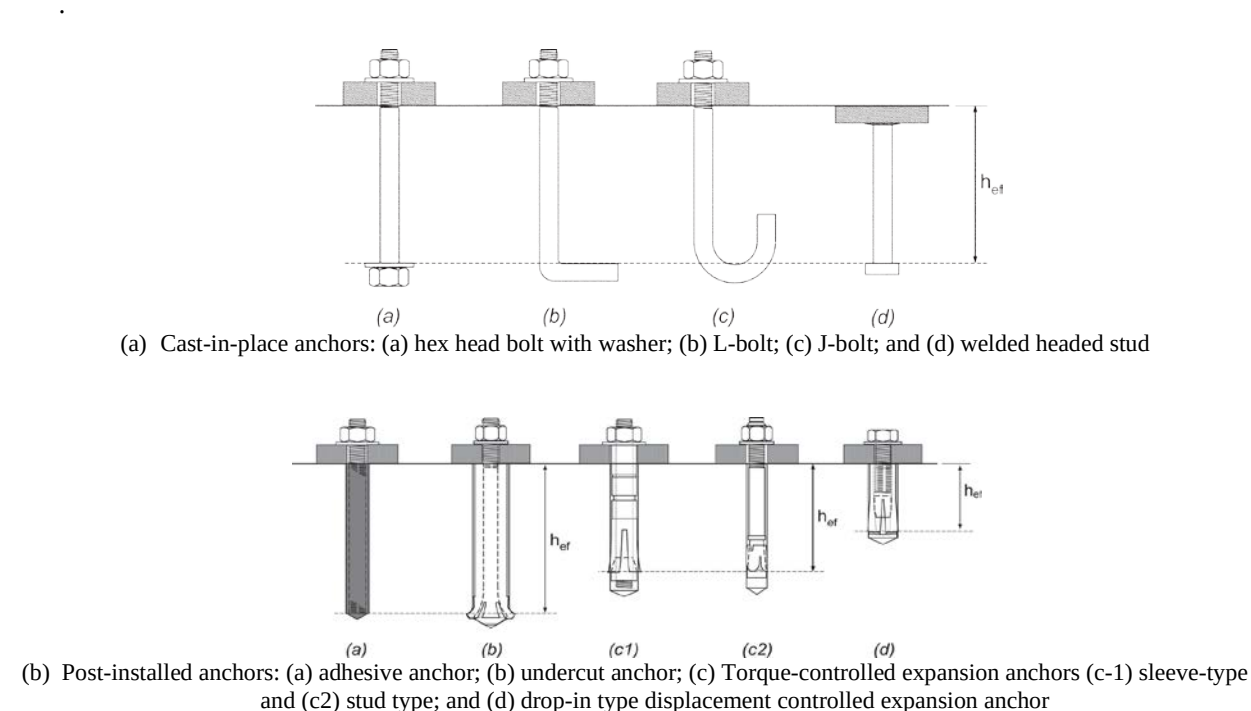


Figure 5—Anchor types included in ACI 318-14 Chapter 17 (Courtesy of ACI)

Failure Modes and Design Concepts

The following failure modes, depicted in Figure 6, shall be considered as applicable:

- Tension
 - Tensile strength of anchor steel
 - Concrete breakout – all types

- o Pullout – cast-in-place, post installed expansion and undercut anchors
 - o Side-face blowout – headed anchors
 - o Bond failure – adhesive anchors
- Shear
 - o Shear strength of anchor steel
 - o Concrete breakout – all types
 - o Pryout – all types
- Tension-shear interaction

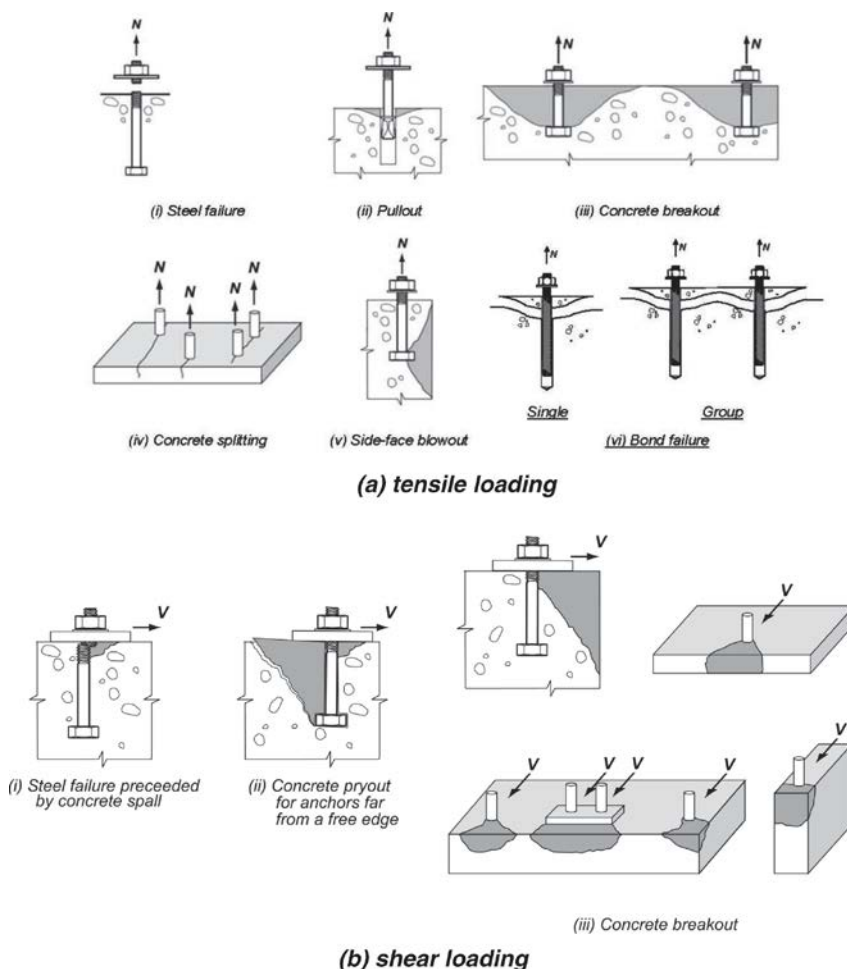


Figure 6—Failure modes considered in ACI 318-14 Chapter 17 (Courtesy of ACI)

Design of steel components, including reinforcing bars, plain bars or threaded bars, against failure in tension, shear or combined shear and tension shall comply with the appropriate provisions of ACI 318-14 Chapter 17 [10] rather than Section 6 of AASHTO LRFD.

Based on a comparison of load factors and resistance factors for concrete design in the ACI and AASHTO specifications it was concluded that the strength reduction factors in ACI 318-14 Chapter 17 [10] are reasonable and possibly somewhat conservative for this first introduction of anchor design into the AASHTO LRFD. Therefore, while “mixing and matching” from different specifications should generally be avoided, the ACI strength reduction factors and models are specified for use in Article 13 rather than the resistance factors specified in Article 5.5.4.2 even though, generally, the loads, load factors and load combinations specified in Section 3 of AASHTO LRFD also apply. Further discussion related to adhesive anchors in particular can be found in Cook et al. [13].

The resistance models for anchors are referenced to ACI [10]. The nominal resistance of cast-in-place anchors is based on calculation procedures in ACI 318-14, Chapter 17 [10] involving 5% fractile strength determined using test methods specified in ACI 355.2 [11]. Similarly, the nominal resistance of post installed anchors is based on 5% fractile determined in accordance with ACI 355.4 [12]. For adhesive anchors for which test data is not available for design there is a table of presumptive minimum characteristic bond stresses which would need adjustments for sustained loading and seismic design.

Sustained tension on Adhesive Anchors

Based on the experience in the 2006 tunnel ceiling collapse in Boston, applications where adhesive anchors are subjected to sustained tension require special mention. Under sustained tension, ACI 318-14 Chapter 17 [10] recommends that less than 55 percent of the resistance be used for a design life greater than 50 years, but no further specific value is suggested. Section 13 requires a 50 percent factor which is based on recommendations for a 100-year service life at 70°F or 20 years at 110°F in Cook et al. [13]. In lieu of owner-supplied criteria, and based on years of successful past practice by one state, significant sustained tensile loads may be considered to be those with an unfactored magnitude exceeding 10 percent of the ultimate capacity of the anchor or anchor group.

Impact Resistance

Impact loading also merits special mention. Safety levels associated with ACI 318-14 Chapter 17 [10] are intended only for in-service conditions rather than short-term handling and construction condition, or for impact or cyclic loads other than those associated with seismic events. These restrictions made applications to some bridge details such as deck mounted appurtenances and hardware problematic. After consultation with researchers and a review of technical literature it was decided that some relief from the ACI restrictions was reasonable and appropriate. Therefore, for the purpose of AASHTO LRFD, anchors attaching pedestrian or bicycle rails or fences separated from the roadway by a traffic barrier meeting the requirements of Section 13 need not be considered subject to the impact exclusion in ACI 318-14 Chapter 17 [10]. Further, the exclusion for impact loads need not apply to other attachments using post-installed anchors shown to have impact strength at least equal to their static strength as documented by testing or a combination of analysis and testing deemed appropriate by the owner. The decision to not require the impact exclusion in AASHTO LRFD for the situations indicated was based on following:

- ACI 349-13 [14] no longer requires the impact exclusion for the design of nuclear facilities. Appendix F contains Dynamic Increase Factors (DIF) for undercut and expansion anchors.
- Shirvani et al. [15] has documentation of similar DIFs for expansion and undercut anchors in tension in both cracked and uncracked concrete.
- Dickey et al. documents the impact behavior of epoxy adhesive anchors and recommends DIFs. The amount of improved strength under impact loads is thought to be dependent on the adhesive used.
- Similar results were reported by Braimah et al. [16].
- Additional references are available in the literature including tests of anchored hardware for traffic barriers and shown in Figure 7 from MwRSF TRP 13-264-12, (Dickey et al, [17]).



Figure 7—Test of anchored barrier support

Even though some research indicates that the impact strength can exceed the static strength, the use of any documented increase in resistance due to impact behavior is left to the discretion of the owner at this time.

Seismic Design

Seismic design is another area where merging ACI and AASHTO requirements requires careful consideration and judgement. The approach taken is explained in the commentary to article 5.13.3 which is excerpted below. The ACI provisions use the Seismic Design Categories (SDC) C through F of ASCE/SEI 7 [18]. SDC C applies to situations with moderate to intermediate seismic risk. SDC D and higher categories apply to high risk situation. A direct correlation between SDC and AASHTO Seismic Zones (SZ) is not straight forward. For example, the generalized shapes of both generalized spectra are quite similar for periods below the ASCE long term period, T_L , but differ beyond that point. Going from a generalized spectrum to a site-specific elastic seismic response coefficient, C_{sm} , requires map data and Site Factors. The ASCE maps are based on a probability of structural failure of approximately once in every 5,000 year while the AASHTO maps present data for a return period of seismic event exceedance of approximately 1,000 years. This difference is partly and nonuniformly compensated by scaling the ASCE Data by $2/3$ developing the Design Response Spectrum. The Site Factors are tabulated by site class and short period acceleration coefficient. The values in the tables are identical, but the selection of which value to use is affected by which set of maps is used to determine the short period acceleration coefficient. Despite these differences, a comparison shows that AASHTO LRFD Seismic Zone 1 reasonably represents ASCE SEI SDCs A and B [18]. There is no distinction in the requirements in ACI 318-14 Chapter 17 for structures in SDC C, D, E or F and therefore no distinction is made in the new provisions among Zones 2, 3 and 4. Thus requirements of ACI 318-14 Article 17.2.3 [10] are used for anchors in the seismic load path on structures in AASHTO LRFD Seismic Zones 2, 3 and 4.

Anchors resisting seismic forces must be suitable for cracked concrete and ACI 355.2 [11] and 355.4 [12] have simulated seismic tests for expansion, undercut and adhesive anchors. These design provisions do not apply in plastic hinge zones due to the extensive cracking and spalling expected in those regions. Where anchors must be located in regions of plastic hinging, they should be designed to transfer load directly to anchor reinforcement designed to carry the anchor forces into the body of the member beyond the anchorage region. Configurations that rely on concrete tensile strength should not be used.

BEAM LEDGES AND INVERTED T-CAPS

Beam ledges are distinguished from brackets and corbels in that their width along the face of the supporting member is greater than $(W + 5a_f)$, as shown in Figure 8. In addition, beam ledges are supported primarily by ties to the supporting member, whereas corbels utilize a strut penetrating directly into the supporting member. Beam ledges are generally continuous between points of application of bearing forces.

Despite the differences between corbels and beam ledges cited above, there are some corbel design provisions that also apply to beam ledges and inverted T-Caps. For example, shear design of the ledge utilized the shear friction provisions of Article 5.7.4 with the nominal interface shear specified for corbels in Article 5.8.4.2.2. Similarly, design of reinforcing A_s shown in Figure 8 for the combine effect of normal force N_{uc} and bending resulting from the normal force and the eccentricity of the vertical reaction V_u is based on the provisions for corbels and is distributed over the width shown in Figure 8 with the caveats that the widths may not overlap.

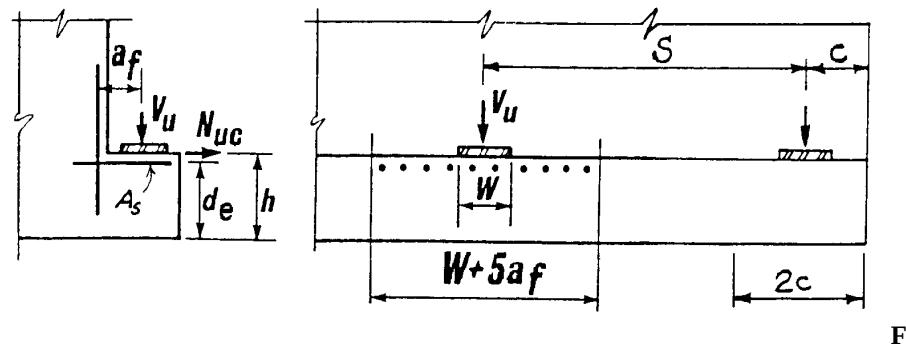


Figure 8—Principal dimensions for beam ledges.

Beam ledges and inverted T-Caps may be designed by either the STM or by a series of provisions based on assumed failure surfaces and corresponding equations. The latter design method is discussed below. Regardless of which design method is used, the specification requires that beam ledges be designed to resist the following failure modes illustrated in Figure 9:

- Flexure, shear, and horizontal forces at the location of Crack 1;
- Tension force in the supporting element at the location of Crack 2;
- Punching shear at points of loading at the location of Crack 3; and
- Bearing force at the location of Crack 4.

While many of the design requirements to provide adequate safety and economy for each of the indicated failure modes have been updated, only revisions to defining the shape of punching shear prism are treated below.

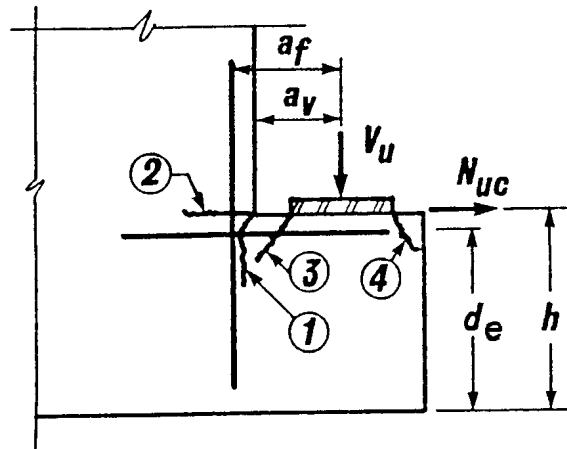


Figure 9—Failure modes for beam ledges\

Punching shear capacity is based on truncated prisms using equations that require that the prisms do not overlap. Equations are provided that assure that overlap is prevented for bearing on opposite sides of inverted T-Caps and for bearing along any type of ledge. If these equations are not satisfied, the prisms overlap and an investigation of the combined surface areas is needed. An important distinction in the revised provisions is that the depth term in the equations, “ d_f ”, is measure from the top of the ledge to the longitudinal cap reinforcement as shown in Figure 10.

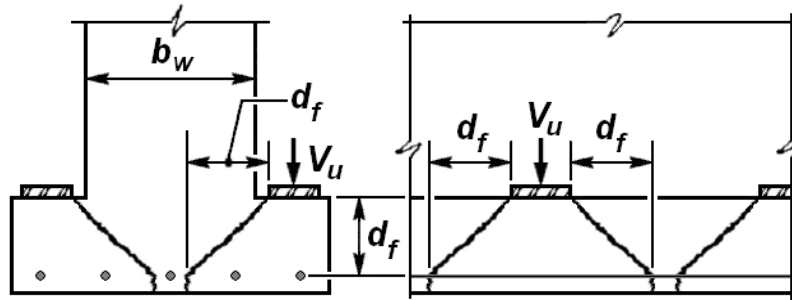


Figure 10—Failure surfaces

The area used to calculate the punching shear capacity is the product of the depth, d_f , and the perimeter of the ledge at approximate mid-depth, b_o , as show in Figures 11 and 12. Equations are provided to determine the perimeter for four cases:

- Interior rectangular pads: $b_o = W + 2L + 2d_f$
- Exterior rectangular pads: $b_o = 0.5W + L + d_f + c \leq W + 2L + 2d_f$
- Interior circular pads: $b_o = \pi/2(D + d_f) + D$
- Exterior circular pads: $b_o = \pi/4(D + d_f) + D/2 + c \leq \pi/2(D + d_f) + D$

Previous provisions used the distance “ d_e ” the distance from the bottom of the ledge to the center of the reinforcement in the top of the ledge. The research supporting the revised provisions used d_f , the “effective flange depth from the top face of the flange to the top of the bottom layer of transverse reinforcement in the flange” (Mirza and Furlong, [19]). Simplifying two equations for punching shear capacity in the previous provisions into a single equation using the term b_o as the length of the perimeter of the failure surface and d_f as the effective depth increases the flexibility of the equation. The punching shear capacity can now be calculated for any bearing shape including circular bearings for which equations for b_o for both interior and exterior bearings have been added. This also matches the calculations for punching (two-way) shear for slabs and footings.

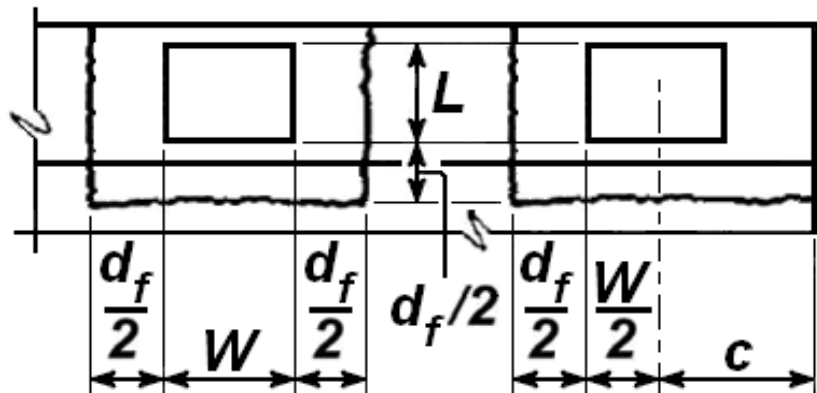


Figure 11—Critical section for rectangular bearing pads

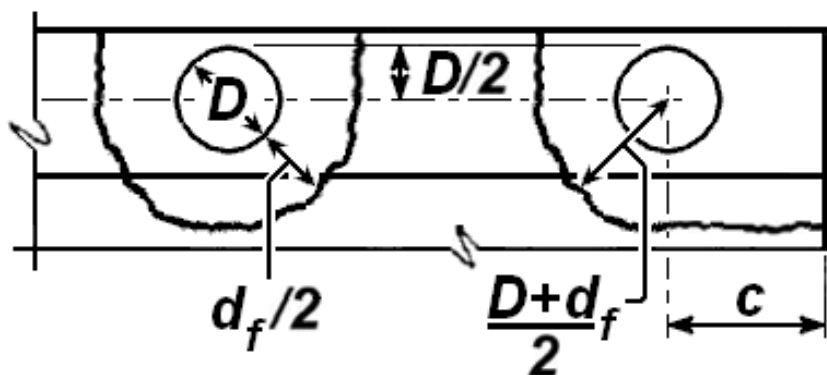


Figure 12—Critical section for circular bearing pads

DURABILITY

Previous provisions relating to durability have been moved to a separate set of articles, 5.14 Durability. The area of bridge design life is receiving more and more focus as owners attempt to specify 100, 150 and even 200 year design life in their contract documents. As on-going research on this topic is completed it was thought that a separate set of provisions on Durability would be the logical place for these new provisions. As the provisions stand today they largely provide practical guidance but it is believed that in the future this will be the place for more prescriptive requirements as owners seek to extend the design service life of bridges.

MISCELLANEOUS CHANGES

In addition to the major changes highlighted above, literally hundreds of local clarifications and corrections have also been made. For example:

- Notation has been culled of symbols no longer used and missing symbols have been added;
- Most articles have some wording changes that make intent clearer e.g.:
 - Conditionals in about 60 locations have been modified to make them easier to understand;
 - Use of the terms bonded, fully bonded, partially unbonded, debonded, partially debonded, blanketed and shielded have been replaced with bonded, debonded and unbonded in about 94 places;
 - A consistent definition of f'_c , “compressive strength of concrete for use in design”, has been introduced in about 390 places, and;
 - Use of the term “non-prestressed reinforcement” instead of “mild reinforcement” as “mild reinforcement” was never defined in previous editions of the Specifications.
- Some articles have been reorganized, streamlined or augmented, e.g. 5.7.4 Interface Shear
- Some individual provisions have been clarified or moved
- Some corrections e.g.
 - combined stress in webs required at the service limit state tension should have been in beta-theta strength limit state calculations;
 - improperly located info has been relocated, e.g. Stirrup spacing commentary in article that did not deal with stirrups, and;
 - Some provisions that had no current function were removed e.g. Article 5.8.4.1 for deep beams made no sense where it was and effectively duplicated the crack control requirements in the strut and tie articles and was replaced with cross ref to STM crack control.
- Some figures that have been a source of confusion have been replaced and some new figures added, and;
- Article 5.12- Provisions for Structure, Components, and Type, formerly Article 5.13, has been reorganized to be more hierarchical and follow a top-down design process.

TRANSITION AIDS

Any one section of the design specification does not live in a vacuum. Other sections of that document, other AASHTO documents and documents prepared by the FHWA, such as training courses, as well as state and association handbooks refer to Section 5 of AASHTO LRFD. In order to assist in the transition to new article numbers and finding relocated content a “Cross walk” table was developed that Lists the old 7th edition article numbers and the new article numbers where that, or similar, material is located. It also indicates whether the material in a given article is unchanged, updated, has editorial changes, or is totally new. Lists of article number changes to other Sections of AASHTO LRFD, the *LRFD Bridge Construction Specifications* [20], the *LRFD Seismic Design Specification* [21], the *Pedestrian Bridge Guide Specifications* [22], and the *Manual for Bridge Evaluation* [23] have also been provided. A downloadable Microsoft Excel spreadsheet version of the crosswalk is available for free from the AASHTO Publications website where the Specifications are available for purchase, as Appendix E5. This tool is intended for those familiar with the 7th Edition of the Specifications to be able to find the corresponding provisions in the 8th Edition. It also allows easy cross reference from the 8th Edition back to the 7th Edition which may prove useful in maintaining historical references to the older edition.

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Load and Resistance Factors for Concrete Bridges and Buildings

Olga Iatsko and Andrzej S. Nowak

Synopsis: In the new generation of design code, safety of structures is provided in form of load and resistance factors. Safety is measured in terms of the reliability index. The acceptability criterion in the selection of load and resistance factors is closeness to the target reliability index which can be different depending on limit state. The paper presents a procedure to determine these factors using the concept of “design point”. The coordinates of design point are equal to factored load or factored resistance. The required input data includes for each load component and resistance: mean values, bias factor (ratio of mean to nominal), standard deviation or coefficient of variation. The procedure is demonstrated on example of bridge design code (AASHTO[1]) and design code for concrete buildings (ACI 318[2]) for prestressed concrete girders and reinforced concrete beams in flexure and shear.

Keywords: load factor, resistance factor, design point, reliability index, dead load, live load, resistance, statistical parameters, bias factor, coefficient of variation.

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INTRODUCTION

A new generation of design codes and guides for evaluation of existing structures is based on consideration of limit states and failure scenarios. Contrary to the allowable stress design (ASD) or working stress design (WSD), safety margin is represented by load and resistance factors. However, the principal role of design codes and guides is to make sure that the designed or evaluated structures perform their function with adequate reliability. It is impossible to eliminate the possibility of failure but the probability of failure should be kept at an acceptably low level. There are three important questions:

- How to measure safety?
- What is acceptable and what is unacceptable safety level?
- How to implement the acceptable safety limit in engineering practice?

There is a considerable literature on structural safety and reliability. Safety can be measured in terms of the reliability index, β . The acceptability/unacceptability criteria can then be expressed as a target reliability index, β_T . For each limit state, the selection of β_T depends on the consequences of failure and marginal cost of safety, and it may involve a higher or lower degree of subjective judgement. However, the objective of this paper is to focus on the last bullet item, i.e. implementation of the target reliability index in design codes and guides for evaluation of existing structures. In particular, the paper deals with derivation of load and resistance factors as coordinates of the so called “design point”[3].

RESEARCH SIGNIFICANCE

Current AASHTO LRFD Bridge Design Specification[1] was calibrated using statistical parameters of load and resistance from the 1970’s and early 1980’s[4]. In the original reliability-based procedure the load factors were calculated assuming that the design factored resistance is located two standard deviations below the mean value. The resistance parameters were calculated using the Rackwitz-Fiesler procedure and previously determined load factors. The acceptability criterion was closeness to the target reliability index ($\beta=3.5$). A similar procedure is applicable to the current ACI 318[2, p. 318]. Parameters of resistance has been updated recently[5], but load and resistance parameters have not yet been updated. To count for the recent updates in the reliability-based code development procedures, load and resistance statistics, the original calibration of AASHTO LRFD[1] and ACI 318[2] design formulas for Strength I Limit state has been re-visited in this study.

ANALITICAL PROCEDURE

Limit state function and reliability index

For each limit state, a structural component can be in two states: safe when resistance, R , exceeds the load, Q , and unsafe (failure) when load exceeds resistance. The boundary between safe and unsafe states can be represented by the limit state function, in a simple form such as:

$$g = R - Q = 0 \quad (1)$$

Where: R – function of resistance.
 Q – function of load.

Since R and Q can be considered as random variables, the probability of failure, P_f , is equal to probability of g being negative,

$$P_f = P(g < 0) \quad (2)$$

In general, R and Q can be functions of several variables such as dead load, live load, dynamic load, strength of material, dimensions, girder distribution factors, and so on. Therefore, the limit state function can be a complex function:

$$g(X_1, \dots, X_n) = 0 \quad (3)$$

A direct calculation of probability of failure can be difficult, in particular when g is non-linear. Instead, a reliability index, β , can be calculated and the relationship between, β , and the probability of failure, P_f , is as follows:

$$P_f = \Phi(-\beta) \quad (4)$$

and

$$\beta = -\Phi^{-1}(P_f) \quad (5)$$

where: Φ = cumulative distribution function of the standardized normal random variable;
 Φ^{-1} = the inverse of Φ [3].

There are several formulas and analytical procedures available to calculate β . If the limit state function is linear, and all the variables are normal (Gaussian), i.e:

$$g(X_1, \dots, X_n) = a_0 + \sum_{i=1}^n a_i X_i \quad (6)$$

where:

$X_1, \dots, X_n$ = input random variables (load and resistance components).
 $a_0 \dots a_i$ = constants of the limit state function.

then,

$$\beta = \frac{\mu_g}{\sigma_g} \quad (7)$$

where:

$$\mu_g = g(\mu_1, \dots, \mu_n) \quad (8)$$

$$\sigma_g = \sqrt{\sum (a_i \sigma_i)^2} \quad (9)$$

g = mean value of g .

$(\mu_1, \dots, \mu_n)$ = mean values of $X_1, \dots, X_n$

μ_g = standard deviation of g .

μ_i = mean value of X_i ,

σ_i = standard deviation of X_i .

If the variables are non-normal, then Eq. 9 can be used as an approximation. Otherwise, a more accurate value of β can be calculated using an iterative procedure or if the limit state function is non-linear, then accurate results can be obtained using Monte Carlo simulations[3].

Design point

Each design provision/formula can be evaluated using the reliability analysis. Therefore, the level of safety is expressed with the reliability index, denoted with, β . Additionally, the result of reliability analysis can be coordinates of the “design point”. The concept of the “design point” was first introduced in 1974 by Hasofer and Lind[6] as a location at the failure surface, $g(R, Q)=0$, where the limit state function can be evaluated, instead of mean values.

For the linear limit state function and normally distributed load and resistance components, the Hasofer-Lind reliability index can be calculated using the First-order, second-moment reliability method[3]. For a two-dimensional representation the corresponding “design point” is located at the coordinates at the intersection of factored load Q^* and factored resistance R^* (Fig. 1). In three-dimensional representation, the location of the design point is shown in Fig. 2.

For the limit state function in Eq. 5, the design point is a point in n -dimensional space, denoted by $(X_1^*, \dots, X_n^*)$, that satisfies Eq. 5, and if failure is to occur, it is the most likely combination of $X_1^*, \dots, X_n^*$ [3]. If the limit state function is given by Eq. 1, and R and Q are normal random variables, then the coordinates of the design point are:

$$R^* = \mu_R - \frac{\beta \sigma_R^2}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (10)$$

$$Q^* = \mu_Q + \frac{\beta \sigma_Q^2}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \quad (11)$$

Where: R^* = coordinate of the design point for R

Q^* = coordinate for the design point for Q

μ_R = mean value of R

σ_R = standard deviation of R

μ_Q = mean value of Q

σ_Q = standard deviation of Q

In most cases the position of the design point is unknown. Thus, the iterative Rackwitz and Fiessler [7] iterative procedure can be applied to determine coordinates of the “design point”, Q^* and R^* . Since a relatively wide range of design point coordinates corresponds to the same value of reliability index, in practice, Eq. 10 and Eq. 11 can be used even for non-normal distributions.

Load and resistance factors

Consider a load and resistance factor design (LRFD) formula,

$$\gamma_D D_n + \gamma_L L_n \leq \phi R_n \quad (12)$$

Where: D_n = nominal dead load,

γ_D = dead load factor,

L_n = nominal live load,

γ_L = live load factor,

R_n = nominal resistance,

ϕ = resistance factor.

Furthermore, $\gamma_D D$ is factored dead load, $\gamma_L L$ is factored live load and ϕR is factored resistance. But the coordinates of design point, D^* , L^* and R^* correspond to factored loads and factored resistance, respectively. Therefore, the load and resistance factors can be calculated as follows,

$$\gamma_D = D^* / D_n \quad (13)$$

$$\gamma_L = L^* / L_n \quad (14)$$

$$\phi = R^* / R_n \quad (15)$$

In many practical cases, to calculate load and resistance factors, the required input data for each load component and resistance includes the following: mean value, bias factor (mean-to-nominal ratio), λ , and standard deviation, σ , or coefficient of variation, V . The derivation of load and resistance factors is demonstrated for bridge design code AASHTO[1], concrete buildings design code ACI 318[2], and a guide for evaluation of existing bridges[8].

CODE CALIBRATION

Bridge design code

The basic load combination for bridge load components include dead load, D , dead load due to the wearing surface, D_w , live load, L , and dynamic load, I . Each random variable is described by its cumulative distribution function (CDF), including the mean and standard deviation. It is also convenient to use the bias factor which is the ratio of mean-to-nominal value, denoted by λ , and the coefficient of variation, V , equal to the ratio of the standard deviation and the mean. Both λ and V are non-dimensional.

The total load is a sum of $D + D_w + (L + I)$. Since there is no study, similar to NCHRP 368[9], conducted for the recent decades, the statistical parameters of dead load component are assumed to remain unchanged. The bias factors and coefficients of variation are summarized in Table 1.

To be consistent, the reliability analysis is performed for the same design cases as in the original NCHRP Report 368[9]. Representative design cases were selected based on the analysis of about 200 bridges designed according to AASHTO (1989) from different regions of the United States, provided by State DOTs [9]. These structures were selected based on types, materials, and span lengths representative for a particular region.

Therefore, for factory-made components (structural steel and precast/prestressed concrete) $\lambda = 1.03$ and $V = 0.08$. For the cast-in-place concrete, $\lambda = 1.05$ and $V = 0.10$. For the wearing surface it is assumed that the mean thickness is 3.5in (90 mm) with $\lambda = 1.00$ and $V = 0.25$. The statistical parameters for live load are taken from the recent SHRP2 R19B report[11]. The ratio of mean-to-nominal value, or bias factor for live load moment, is plotted vs. span length in Fig. 3 for the average daily truck traffic (ADTT) from 250 to 10,000. The mean value of the dynamic load factor is taken as 0.10 and the resultant coefficient of variation for static and dynamic live load is determined as 0.14. The total load as a sum of several components can be considered as a normal random variable. The load carrying capacity is considered as a product of three factors representing the uncertainties involved in material properties, dimensions/geometry and the analytical model. The statistical parameters, bias factor, $\lambda = 1.05$, and coefficient of variation, $V = 0.075$, that were used in the original calibration.

The resultant reliability indices for prestressed concrete AASHTO girders for flexure are shown in Fig. 4. In average, the reliability index is close to 3.5, which is considered as a target reliability index, β_r , for structural bridge components. A small degree of variation indicates that the code is consistent.

The dead load factors γ_D corresponding to the design point is shown in Fig. 5 for prestressed AASHTO girder and summarized in Table 2. From Fig. 5 it is clear that the values of the dead load factors are mostly consistent with regard to different span lengths and ADTT. The distribution of the live load factor, in its turn, is substantially more sensitive to the ADTT, material, and length of the girder. The calculated live load factors are shown in Fig. 6 and summarized in Table 3. For most cases, the optimum live load factor γ_L is between 1.4 and 1.55 for ADTT = 10,000 and the range is 1.3 to 1.5 for ADTT = 250. Therefore, 1.55 can be considered as a conservative value of live load, even for ADTT = 10,000. The resistance factors are presented in Fig. 7 for prestressed concrete girders.

Design code for Concrete Structures

The statistical parameters for the design of reinforced concrete building structures can be different than for bridges. The resultant load and resistance factors primarily depend on statistical parameters of the variables in the particular design formula and proportion of the dead load component, D , and live load, L . The dead load can be assumed the same as listed for bridges (Table 2). However, for live load the bias factor and coefficient of variation depends on the influence area, where live load is distributed. In this study the influence area of 400ft² (37.16m²) was selected. Therefore, the corresponding statistical parameters are $\lambda = 1.00$ and $V = 0.18$. This tributary area is selected for demonstration purposes to compare the results with the corresponding values described by Nowak et al. [5]. Considering statistical parameters of 200ft² ($\lambda = 1.00$ and $V = 0.2$) resulted in 5% reduction of reliability index. Coefficient of variation of load collected from influence area from 1,000 to 10,000ft² varies from 14 to 10%. The resultant reliability index and, load and resistance factors are less conservative than presented in this paper. The ratio of $D/(D+L)$ varied from 0 to 1.0 to cover the extreme cases, where dead, D , or live load, L , component is strongly dominating.

For resistance representing moment carrying capacity of a reinforced concrete beam, two cases of reinforcement ratio are considered[5] for $\rho = 0.60\%$ ($\lambda_{0.6\%} = 1.14$ and $V_{0.6\%} = 0.08$) and $\rho = 1.60\%$ ($\lambda_{1.6\%} = 1.13$ and $V_{1.6\%} = 0.085$). The resulting reliability indices are shown in Fig. 8. They vary from 4.3 for most of the cases to 3.5 when the case 1.4D becomes governing in the design equation. The resultant load and resistance factors are shown in Fig. 9, Fig. 10 and Fig. 11 respectively. The recommended load and resistance factors are,

$$1.3 D + 1.6 L < 0.9 R \quad (17)$$

For comparison, the current ACI 318-14 design formula is:

$$1.2 D + 1.6 L < 0.9 R \quad (18)$$

Updated Load Factors for Design of Concrete Structures

The distribution of reliability index clearly demonstrates a drop/an inconsistency at $D/(D+L)$ from 0.6 to 1.0 for both cases of reinforcement. This trend is further reflected in the plots of resistance factors (Fig. 9), dead load factors (Fig. 10) and live load factors (Fig. 11). The existent load combinations do not properly cover the

range of cases where the dead load component starts dominating in the design equation. Therefore, it is reasonable to change the current design load combination from $1.4D$ to $1.4(D+L)$ to more efficiently count for all of the cases of $D/(D+L)$.

There are three tandem combinations considered in this study:

- The most severe of $1.2D+1.6L$ and $1.4D$
- The most severe of $1.2D+1.6L$ and $1.4(D+L)$

These two cases were applied for the design equation of the concrete members with ratio of reinforcement $\rho=0.6\%$ and $\rho=1.6\%$. As an example, the resultant reliability indices, load and resistance factors are shown in Fig. 12, Fig. 13, Fig. 14 and Fig. 15. The new load combination $1.4(D+L)$ start controlling at $D/(D+L)=0.5$, where the proportion of the dead load, D , and live load, L , is equal and make the rest of distribution curve more uniform.

Therefore, the recommended load combination for the design of reinforced concrete beams is:

$$1.2D+1.6L \text{ and } 1.4(D+L) \quad (19)$$

CONCLUSION

In the new generation reliability-based design codes, the load and resistance factors correspond to the coordinates of the design point. The presented procedure results in a set of load and resistance factors that provide a closer fit to the target reliability index.

Recommended load combinations for the design of concrete structures cover the range of self-weight dominated concrete beams.

The procedure is demonstrated on a prestressed concrete girder and a reinforced concrete beam. For both components, the load and resistance factors are derived for a new design.

NOTATION

Φ = cumulative distribution function of the standardized normal random variable;

Φ^{-1} = the inverse of Φ [3].

β = reliability index.

R = function of resistance.

Q = function of load.

μ_i = mean value of X_i

σ_i = standard deviation of X_i .

$X_1, \dots, X_n$ = input random variables (load and resistance components).

$a_0 \dots a_i$ = constants of the limit state function.

g = mean value of g .

$(\mu_1, \dots, \mu_n)$ = mean values of $X_1, \dots, X_n$

μ_g = standard deviation of g .

μ_i = mean value of X_i ,

σ_i = standard deviation of X_i .

μ_R = mean value of R

σ_R = standard deviation of R

R^* = coordinate of the design point for R

Q^* = coordinate for the design point for Q

μ_Q = mean value of Q

σ_Q = standard deviation of Q .

D_n = nominal dead load,

γ_D = dead load factor,

L_n = nominal live load,

γ_L = live load factor,

R_n = nominal resistance,

ϕ = resistance factor.

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Table 1-Statistical Parameters of the Dead Load Components

Component of dead load	Bias factor, λ	Coefficient of variation, V
D_1 , weight of factory-made elements	1.03	0.08
D_2 , weight of cast-in-place concrete members.	1.05	0.10
D_w , weight of the wearing surface	1.00 (3.5in)	0.25

Table 2: Updated Load Factors for Dead Load Components - Flexure

Dead Load Component	Calculated	Optimum	Recommended
γ_{D1} , (weight of factory made elements)	1.05-1.10	1.15	1.20
γ_{D2} , (weight of cast-in-place concrete members)	1.10-1.17		
γ_{Dw} , (weight of the wearing surface)	1.03-1.10		

Table 3- Updated Load Factors for Live Load Component - Flexure

Material	Live Load Factor γ_{LL} in Current AASHTO LRFD	Calculated Live Load Factor, γ_{LL}	Optimum Live Load Factor, γ_{LL}	Recommended Live Load Factor, γ_{LL}
Composite and Non-composite steel	1.75	1.35-1.65	1.55	1.60
Prestressed concrete		1.35-1.60		
Reinforced concrete		1.30-1.55		

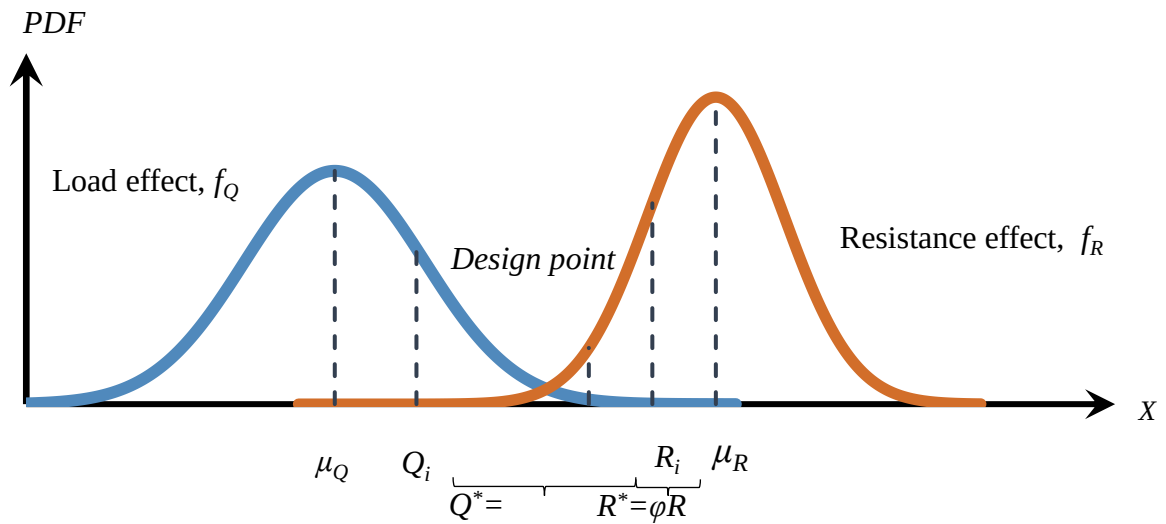


Fig. 1 – Design point on PDFs of load and resistance for linear function $g(R, Q) = R - Q$

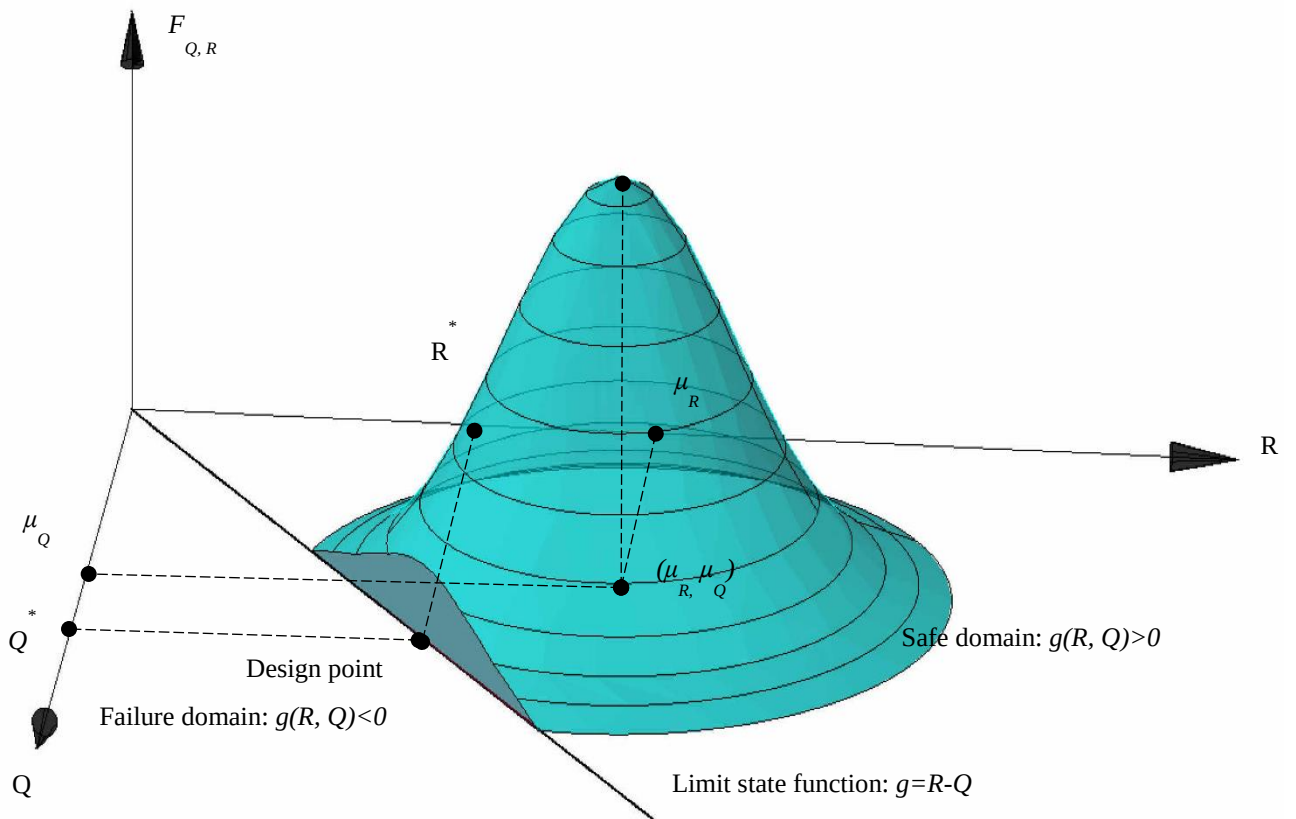


Fig. 2 – Design point at the failure boundary for linear function $g(R, Q) = R - Q$

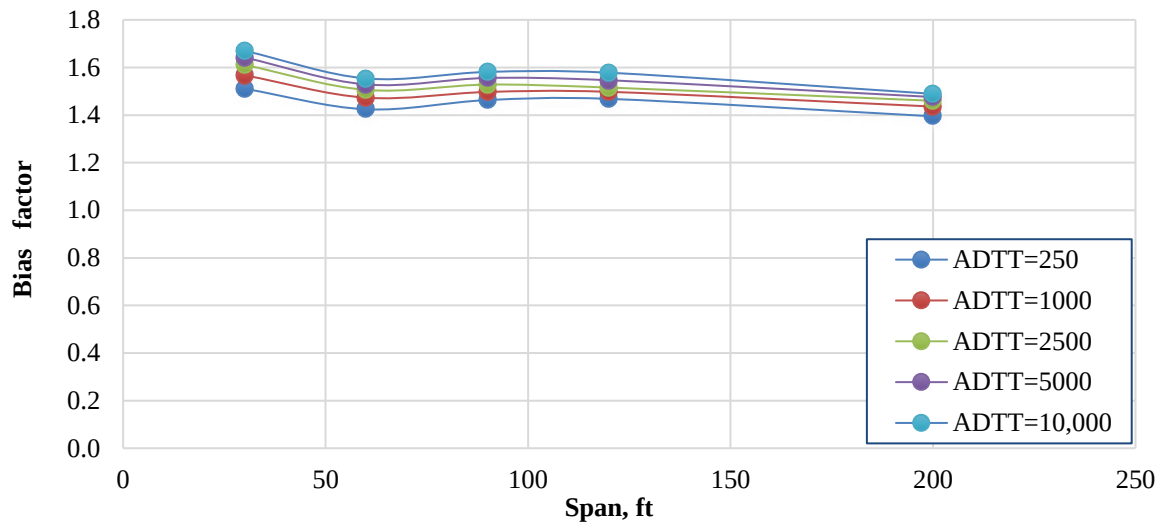


Fig. 3 – Bias factor vs. Span Length for the Moment

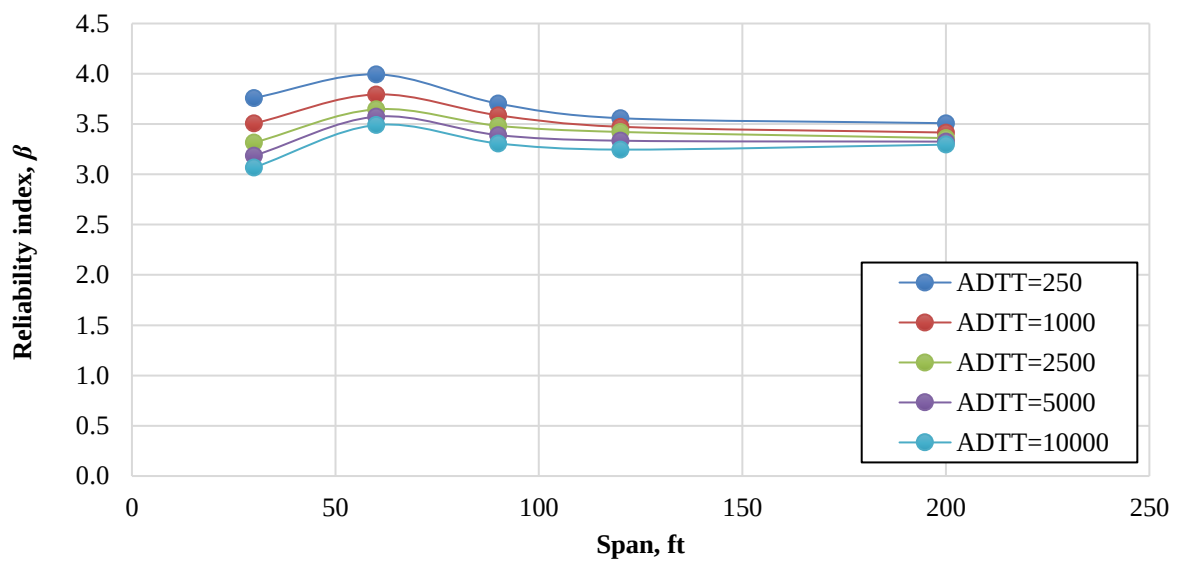


Fig. 4 – Reliability Index vs. Span Length for Moment for Prestressed Concrete Girders

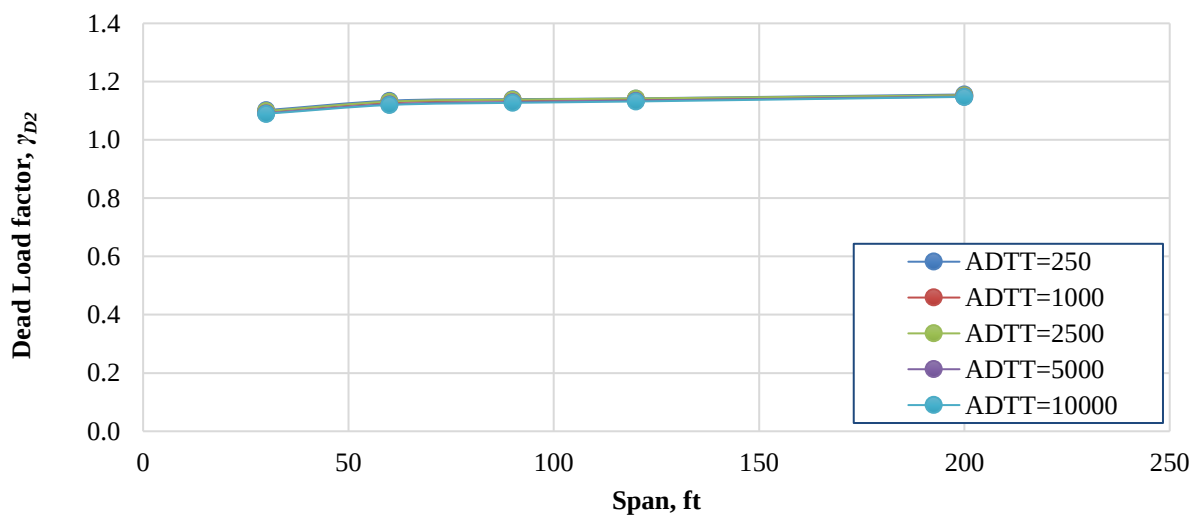


Fig. 5 – Dead Load Factors vs. Span Length for Prestressed Concrete Girders

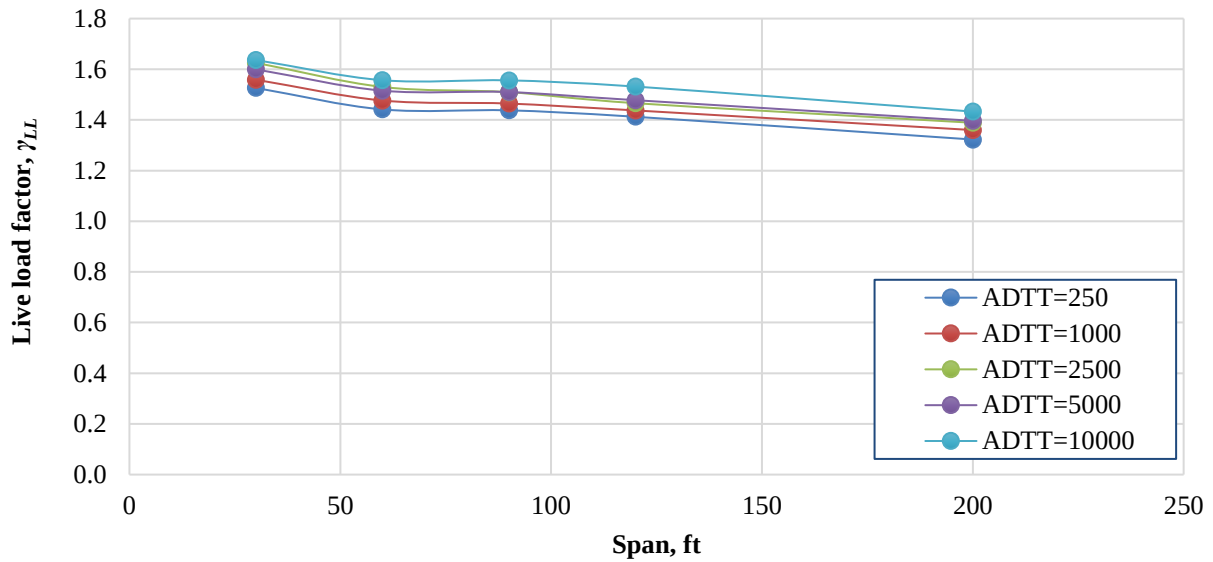


Fig. 6 – Live load factor vs. Span Length for Prestressed Concrete Girders

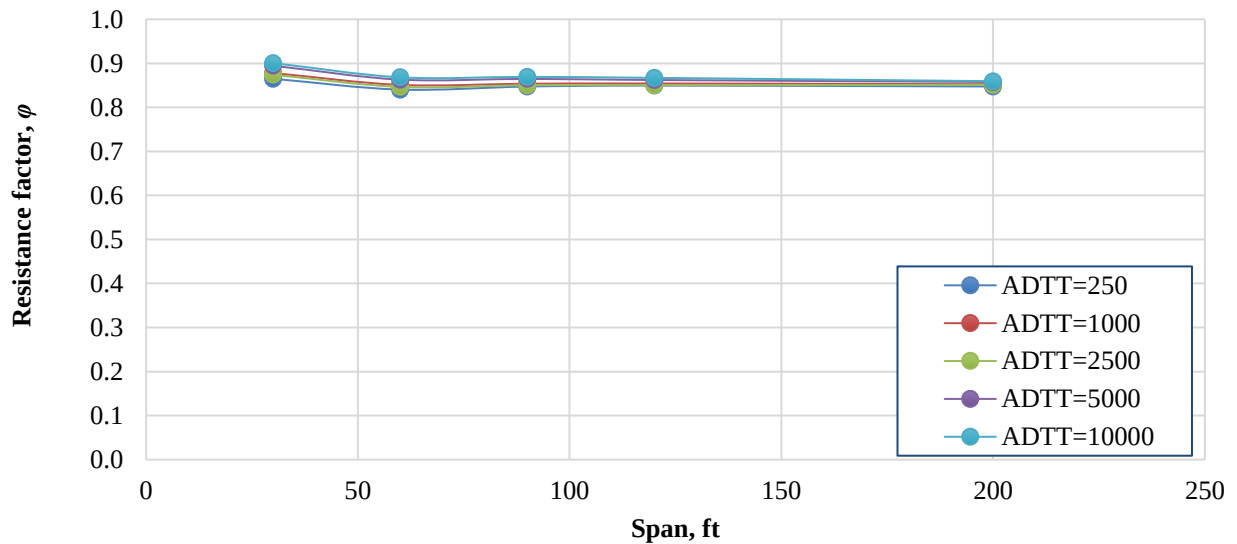


Fig. 7 – Resistance Factor vs. Span Length for Prestressed Concrete Girders

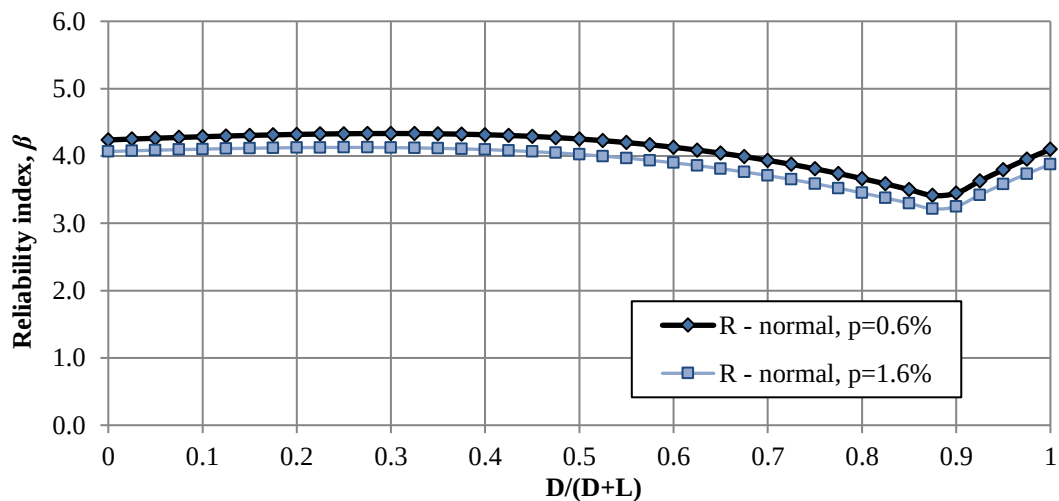


Fig. 8 – Reliability index for design of concrete beams

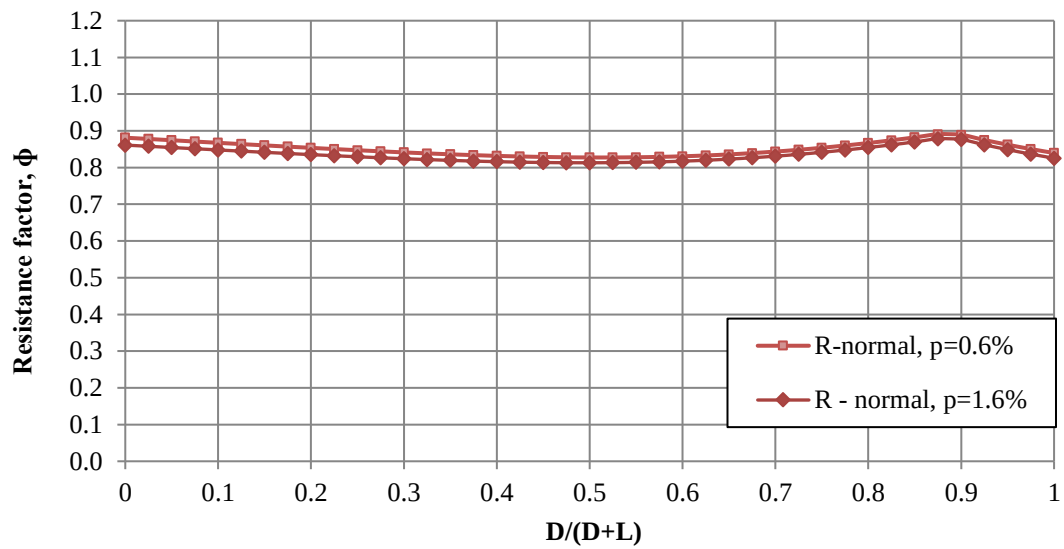


Fig. 9 – Resistance factor for design of concrete beams

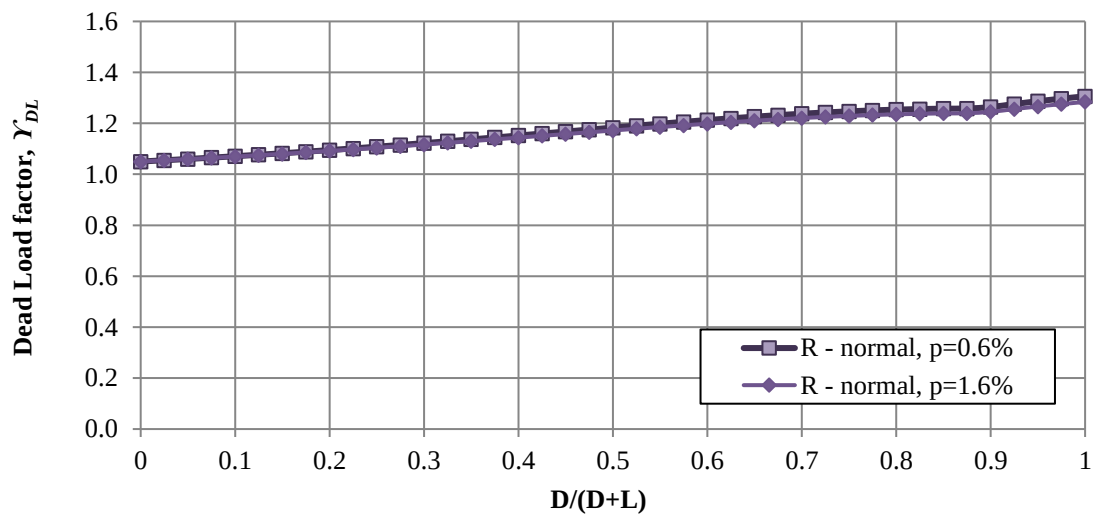


Fig. 10 – Dead load factor for design of concrete beams

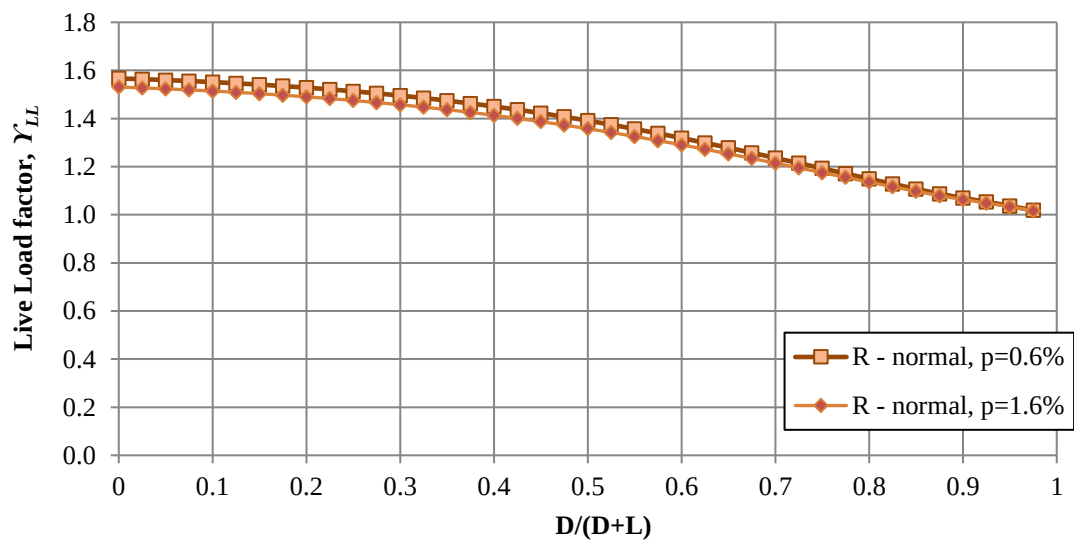
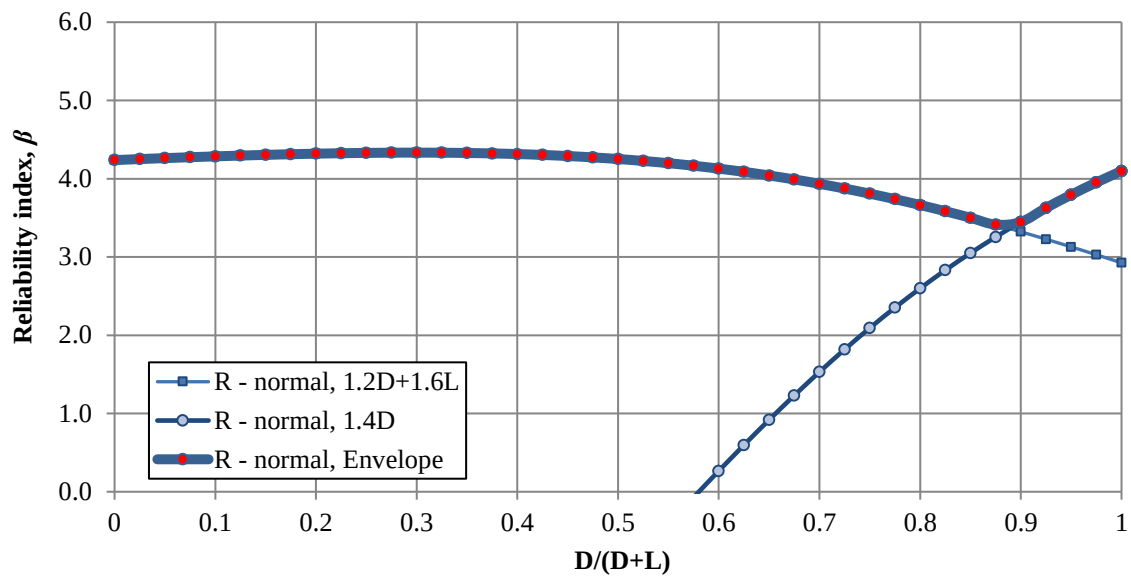
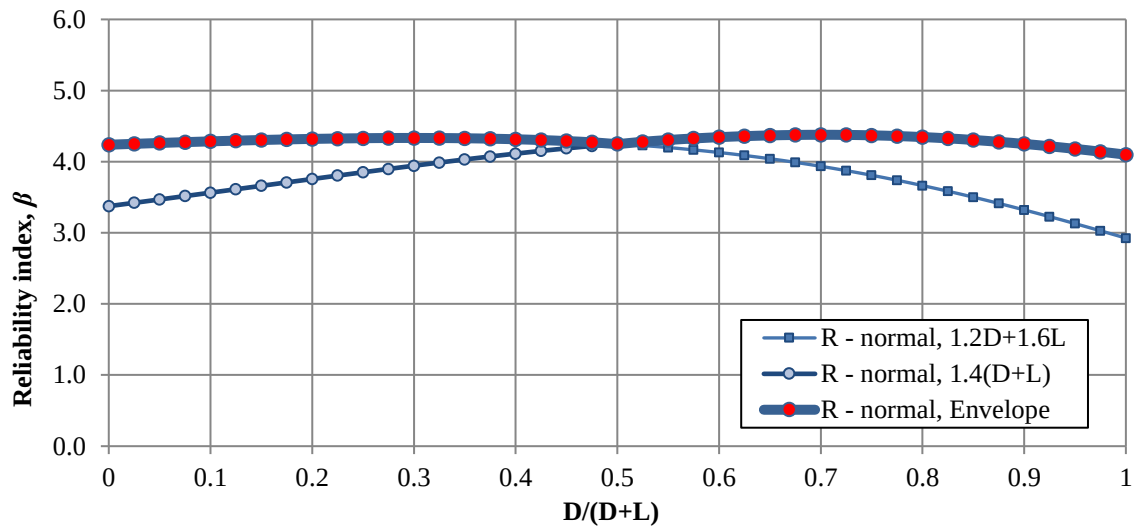


Fig. 11 – Live load factor for design of concrete beams



a)



b)

Fig. 12 – Reliability index for design of concrete beams with $\rho=0.6\%$ for case a) 1.4D and b) 1.4(D+L)

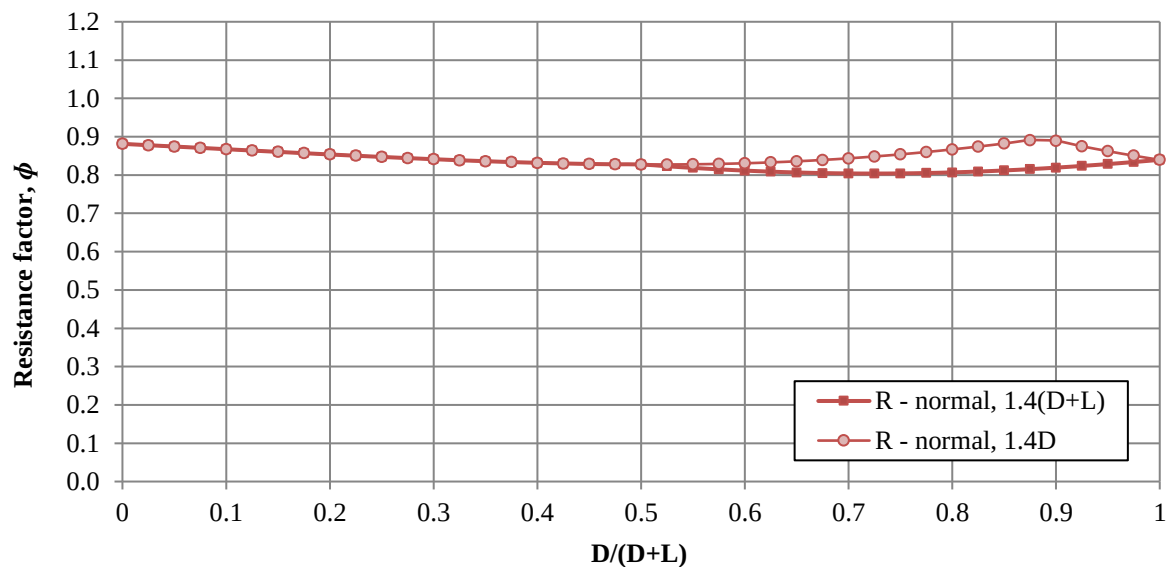


Fig. 13 – Resistance factor for design of concrete beams with $\rho=0.6\%$ for 1.4D and 1.4(D+L)

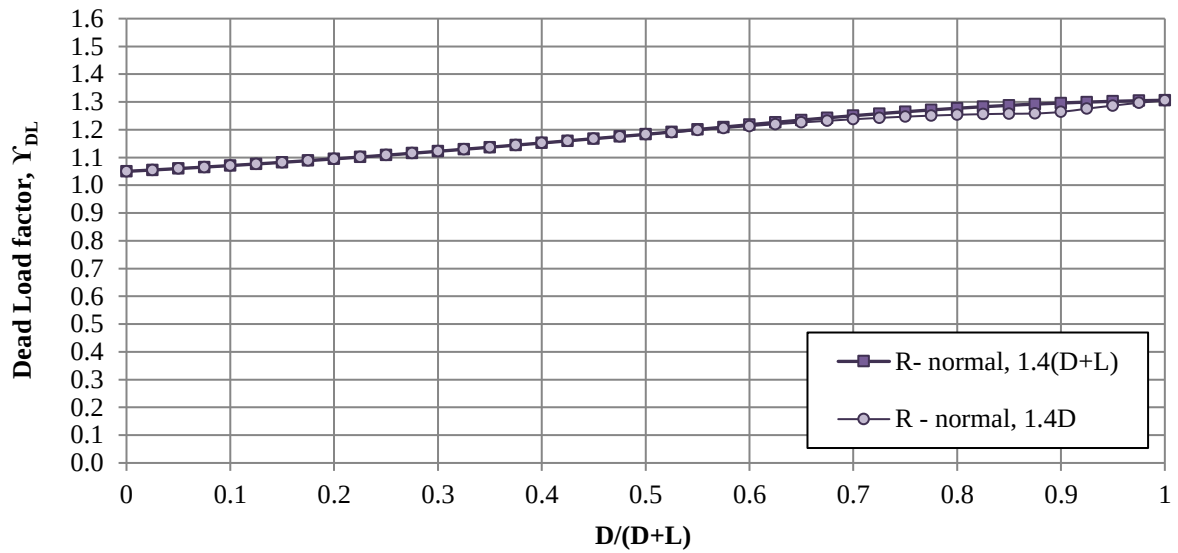


Fig. 14 – Dead load factor for design of concrete beams with $\rho=0.6\%$ for 1.4D and 1.4(D+L)

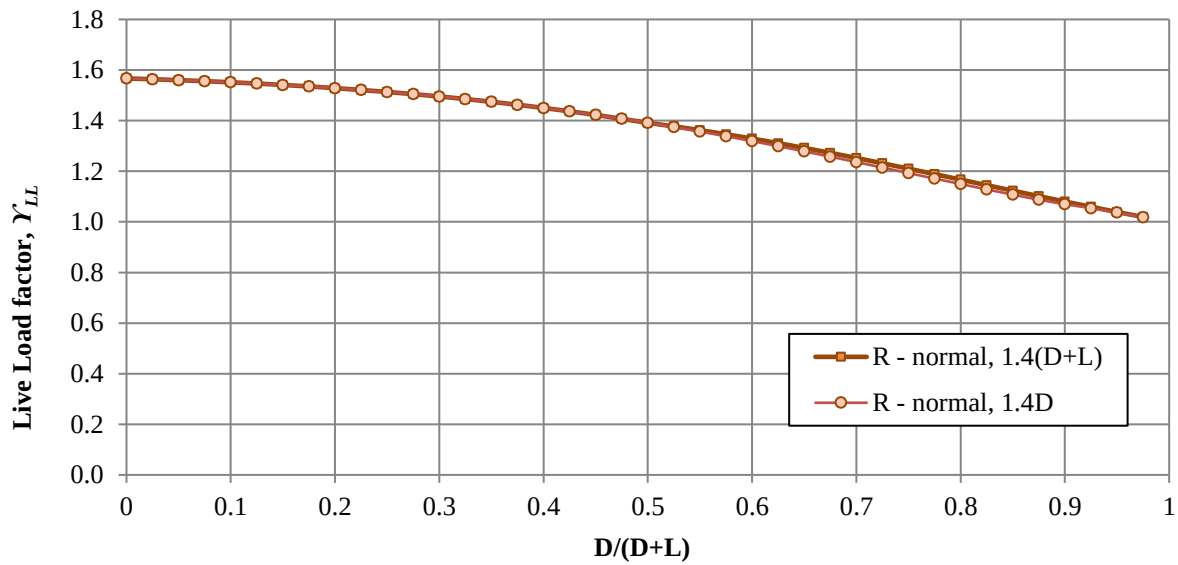


Fig. 15 – Live load factor for design of concrete beams with $\rho=0.6\%$ for 1.4D and 1.4(D+L)

Calibration of Service I Limit State for Reinforced Concrete Bridge Deck Designed using AASHTO Empirical Method

Dan Su and Hani Nassif

Synopsis: Service I limit state in the AASHTO LRFD Bridge Design Specifications (BDS) is applied for the control of cracking in reinforced concrete elements in order to maintain its normal functionality and to achieve its design life. There are two methods specified in AASHTO LRFD BDS: 1) equivalent strip design method and 2) empirical method. For the empirical method, no exhaustive design calculation are needed and the reinforcement area is obtained as a percentage of the concrete section. However, usually, the reinforcement area designed using empirical method is less than that designed using the equivalent strip method, which could result in shortened service life and excessive crack width. Albeit arching action effects were considered in the empirical method which improves the flexural resistance of concrete deck after cracking, the effects of arching action on crack control of reinforced concrete deck were not studied. In addition, different exposure conditions and different design sections (positive moment vs. negative moment regions) were not considered in the empirical design method. Thus, it is extremely important to investigate and calibrate the Service I limit State for reinforced concrete decks designed using the AASHTO empirical method.

In this study, the Service I limit state function is formulated and the load and resistance models are developed. The arching action effects are integrated into the resistance model. Detailed calibration is performed to ensure uniform target reliability will be achieved for different design parameters including exposure conditions, span lengths, deck thickness, and positive moment and negative moment regions.

Keywords: code calibration; limit state; reinforced concrete deck; crack control

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INTRODUCTION

Although the ultimate limit state, such as flexural capacity, shear, or stability, governs the overall safety of the structure, the accumulated damage due to cracking, excessive deflections, and vibrations would also lead to limitation of structural performance, abbreviation of service life, as well as extensive maintenance cost. The current AASHTO LRFD BDS was developed based on structural reliability approach and provided a uniform reliability at strength limit state. In the NCHRP 12-83 project by Wassef et al. (2014), various serviceability limit states were developed and calibrated including Service I and Service III limit states. However, for the calibration of Service I limit state covered the reinforced concrete deck designed using the AASHTO equivalent strip design method only. The empirical method as specified in the AASHTO BDS for the design of reinforced concrete decks, also needs to be calibrated to be consistent with the equivalent strip design method. Thus, in this paper, the authors aim to investigate and calibrate the Service I limit state for reinforced concrete decks designed using the Empirical Design (ED) method. The limit state function is formulated, and the load and the resistance models are developed. The arching action effects were integrated into the resistance model and detailed calibration was performed to ensure uniform target reliability was achieved for different design parameters including exposure conditions, span lengths, deck thickness, and positive moment and negative moment regions.

Over the years, many studies have been done to (1) investigate the critical parameters and their effects on the cracking of reinforced concrete structures, and (2) develop crack width prediction equation based on the design of the deck and environmental conditions. One of the early studies by Clark (1956) included testing 58 specimens and collecting over 105 crack width readings. Clark (1956) concluded that the average crack width is closely related to the following parameters: 1) the diameter of the reinforcing bar, 2) total area of reinforcing bars 3) area of the beam section, and 4) the distance from the bottom reinforcement to the beam bottom surface (i.e. cover). Moreover, Clark (1956) stated that the average crack width was also proportional to the stresses in the reinforcing bars beyond the concrete cracking stress. Clark (1956) suggested that the width of the cracks can be reduced by using a large number of small diameter bars and by increasing the ratio of the steel reinforcement. Kaar and Mattock (1963) developed a well-known crack width equation for the cracked bottom face. Broms (1965) conducted tests on 37 tension and 10 flexural members to analyze crack width and crack spacing. Broms (1965) observed that the crack spacing decreased rapidly with increasing load and

that a number of primary tensile cracks formed on the surface of flexural and tension members. Secondary tensile cracks were confined to the surrounding area of reinforcement. The study by Broms (1965) concluded that the absolute minimum visible crack spacing is the same along the distance from the surface to the center of the reinforcing bar located nearest to the surface of the member. Thus, the theoretical minimum crack spacing is equal to the thickness of the concrete cover (Broms, 1965). Additionally, Gergely and Lutz (1968) developed an equation to predict the crack width based on a detailed statistical assessment of experimental data available in the literature at the time. Gergely and Lutz (1968) identified various parameters, such as reinforcing bar locations, stresses in the reinforcement, concrete cover depth, and spacing of the reinforcement as the controlling factors affecting the crack width. The maximum concrete cover tested in this study was 3.31 inches. However, only three test specimens with cover greater than 2.5 in. were tested in the study. In the study by Frosch (1999), the crack widths were determined from an equation developed based on a physical model. Results were compared with the test data used in Kaar and Mattock (1963) and Gergely and Lutz (1968). The crack width model developed in this study showed that the crack spacing and width are functions of the distance between the reinforcing steel. Crack control can be achieved by limiting the spacing of these reinforcing bars. Based on the research findings, Frosch (1999) suggested that limiting the maximum bar spacing would prevent large cracks in the concrete beams. Frosch (2001) summarized the physical model for cracking and illustrated the development and limitations of the proposed design method. Based on the physical model, Frosch (2001) recommended formulas for calculating the maximum crack width for uncoated and epoxy-coated reinforcement, as well as the design recommendation for their use similar to those presented in Frosch (1999). Table 1 shows a summary of maximum crack width equations developed over the years by various researchers.

Table 1. Equations of Maximum Crack Width

Author	Maximum Crack Width Equation	Year
Clark	$w_{ave} = C_1 \frac{D}{p} [f_s - C_2 (\frac{1}{p} + n)]$	1956
Kaar-Mattock	$w_b = 0.115 \beta f_s \sqrt[4]{A}$	1963
Broms	$w_{ave} = 2t \varepsilon_s$	1965
Gergely-Lutz	$w_b = 0.076 \beta f_s \sqrt[3]{Ad_c}$	1968
Beeby	$w = (K_1 c + K_2 \frac{\phi}{\rho}) \varepsilon_m$	1979
CEB/FIP	$w_k = l_{s,max} (\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs})$	1990
Eurocode	$w_k = S_{r,max} (\varepsilon_{sm} - \varepsilon_{cm})$	1991
Frosch	$w_c = \frac{2 f_s}{E_s} \beta \sqrt{(d_c^2 + (\frac{s}{2})^2)}$	1999

w_{ave}	= average width of cracks, in.
C_1, C_2	= coefficients that depend on distribution of bond stress, bond strength, and tensile strength of concrete, for Clark's study; $C_1 = 2.27 \times 10^{-8} (h-d) / d$, $C_2 = 56.6$.
D	= diameter of reinforcing bar, in.
p	= A_s / A_e = cross-sectional area of reinforcement / cross-sectional area of concrete.
A	= average effective concrete area around reinforcing bar, having same centroid as reinforcement, in. ²
t	= thickness of the concrete cover.
ϵ_s	= the average steel strain.
w	= crack width, in.
K_1, K_2	= constants based on experimental results.
c	= concrete cover.
ϵ_m	= average strain.
ϕ	= bar diameter, in.
ρ	= reinforcement ratio.
w_k	= characteristic crack width.
$l_{s,max}$	= the maximum distance over which slip between the concrete and steel occurs.
ϵ_{sm}	= average steel strain within $l_{s,max}$.
ϵ_{cm}	= average concrete strain within $l_{s,max}$.
ϵ_{cs}	= concrete shrinkage strain (would be a negative value).
$S_{r,max}$	= maximum crack spacing.
A_e	= bd , in. ²
f_s	= computed stress in reinforcement, psi.
n	= ratio of modulus of elasticity of steel to concrete (assumed to be 8 in Clark's study)
h	= overall depth of beam/slab, in.
d	= distance from compressive face of beam/slab to centroid of longitudinal tensile reinforcement.
w_b	= maximum crack width, 0.001 in.
β	= ratio of distances to neutral axis from extreme tension fiber and from centroid of reinforcement. (Kaar and Mattock, 1963, Gergely and Lutz, 1968)
β	= $1.0 + 0.08 d_c$ (Frosch, 1999)
d_c	= bottom cover measured from center of lowest bar, in.
s	= maximum permissible bar spacing, in.
w_c	= limiting crack width, in. (0.016 in, based on ACI 318-95).
E_s	= elastic modulus of steel reinforcement (can be taken as 29000 ksi.)

Crack control in concrete structures has been debated with regard to crack width and its effect on durability and aesthetics. Cracking is primarily caused by flexural and tensile stresses, but may also result from temperature, shrinkage, shear, and torsion. Although researchers do not agree on any one specific crack width equation, the most significant parameters to control cracking have been widely agreed upon. The most sensitive factors are the reinforcing steel stress, followed by concrete cover, bar spacing, and the area of concrete surrounding each bar. For engineering practice, equations in the ACI 318-14 Code and AASHTO LRFD (2017) Specifications are used to control cracking. The corresponding provisions are discussed below.

The ACI requirements for flexural crack control in beams and one-way slabs are based on the statistical analysis of maximum crack width data from several sources (Gergely and Lutz 1968). ACI Committee 224 (2008) provides comprehensive information on control of cracking in concrete structures. ACI maintains that crack control is particularly important when reinforcement with yield strength over 40,000 psi is used. Good detailing practices, such as concrete cover and spacing of reinforcement, should lead to adequate crack control—even when reinforcement strength in excess of 60,000 psi is used. ACI 318-14 Section 24.3.2 does not make a distinction between interior and exterior exposure since corrosion is not clearly correlated with surface crack widths in the range normally found with reinforcement stresses at service load levels. ACI 318-14 only requires that the spacing of reinforcement closest to a surface in tension, s , shall not exceed the value given by the following relationship:

$$s = 15 \left(\frac{40,000}{f_s} \right) - 2.5c_c \quad \text{Eq. 1}$$

$$\text{but not greater than } 12 \left(\frac{40,000}{f_s} \right)$$

where c_c is the least distance from the surface of the reinforcement or prestressing steel to the tension face. If there is only one bar or wire nearest to the extreme tension face, the s used in Eq. (1) becomes the width of the extreme tension face. These provisions are not sufficient for structures subject to very aggressive exposure or designed to be watertight. Special investigation is required for structures subject to very aggressive exposure or designed to be watertight. ACI 318-14 recommends the use of several bars at moderate spacing rather than fewer bars at larger spacing to control cracking.

AASHTO LRFD (2017) also provides provisions for reinforcement spacing to control flexural cracking. Similar to the equation adopted in ACI, AASHTO emphasizes the importance of reinforcement detailing and the notion that smaller bars at moderate spacing are more effective than larger bars of equivalent area. Article 5.6.7 (formerly 5.7.3.4) addresses the distribution of tension reinforcement to control flexural cracking for all concrete components, except deck slabs designed in accordance with Article 9.7.2: Empirical Design. The article requires that the spacing, s , of non-prestressed reinforcement in the layer closest to the tension face shall satisfy the following equation:

$$s \leq \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c \quad \text{Eq. 2}$$

where,

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)} \quad (\text{geometric relationship between crack width at tension face versus crack width at reinforcement level})$$

γ_e = exposure factor = 1.00 for Class I exposure, 0.75 for Class II exposure

d_c = thickness of concrete cover measured from extreme tension fiber to center of the flexural reinforcement located closest thereto (in.)

f_{ss} = tensile stress in steel reinforcement at the service limit state (ksi)

h = overall thickness of depth of the component (in.)

Unlike ACI, AASHTO specifies exposure conditions to meet the needs of the authority having jurisdiction. Class I exposure condition is based on a crack width of 0.017 in. and applies when cracks can be tolerated due to reduced concerns of appearance and/or corrosion. This exposure class can be thought of as an upper bound with regards to crack width for appearance and corrosion. Class II exposure condition applies to transverse design of segmental concrete box girders for any loads applied prior to attaining full nominal concrete strength. This is used when there is an increased concern of appearance and/or corrosion such as in substructures exposed to water and decks. AASHTO LRFD also specifies requirements for skin reinforcement based on ACI 318-02. The AASHTO Equation, shown above in Eq. 2, also applies to both reinforced and prestressed concrete with specifications on the steel stresses used. In general, if AASHTO Class II exposure condition is used, all AASHTO spacings were less than those derived by the ACI equation. However, if Class I exposure condition is used, ACI spacing becomes more conservative.

Cracking of structures is rather common and is not always damaging to the structure. However, when considering a bridge deck, moderately sized cracking can be detrimental to the longevity of the structure due to the exposure to harsh environments. In this study, limit state function was formulated for Service I limit state for reinforced concrete deck designed using empirical method, target reliability index was determined based on reliability evaluation of current design practices, and the calibration was conducted to achieve a consistent target reliability level.

FORMULATION OF LIMIT STATE FUNCTION

Crack control of reinforced concrete deck is an important issue for the serviceability and durability of concrete deck. The crack opening leads to the corrosion of reinforcement, and then further affects the rigidity and duration of the structure. In the general form, a limit state function can be formulated as $G = R - Q$, where G is the limit state function, R is the resistance, and Q is the load. Regarding crack control of reinforced concrete deck, the maximum crack width of reinforced concrete member can be expressed as a function of stress in steel reinforcement, elastic modulus of steel and geometrical parameters such as reinforcement spacing, and concrete cover. Among these variables, stress in steel reinforcement is the only load-driven variable that indicates the relationship between the crack width and the applied live load. Therefore, the load and resistance is expressed as the stress in the steel reinforcement in the limit state function. As shown in Figure 1, 4 maximum crack width prediction equations were compared with experimental data collected by Clark (1956). It is observed that all the

equations used to predict the maximum crack width has exhibited similar margin of error between 20-30%. Since the equation developed by Frosch (1999) was based on a physical model and it is also the basis for the ACI 318-19 as well as the AASHTO LRFD BDS (2017) provisions. Thus, the maximum crack width equation developed by Frosch (1999) is selected to be used in this study.

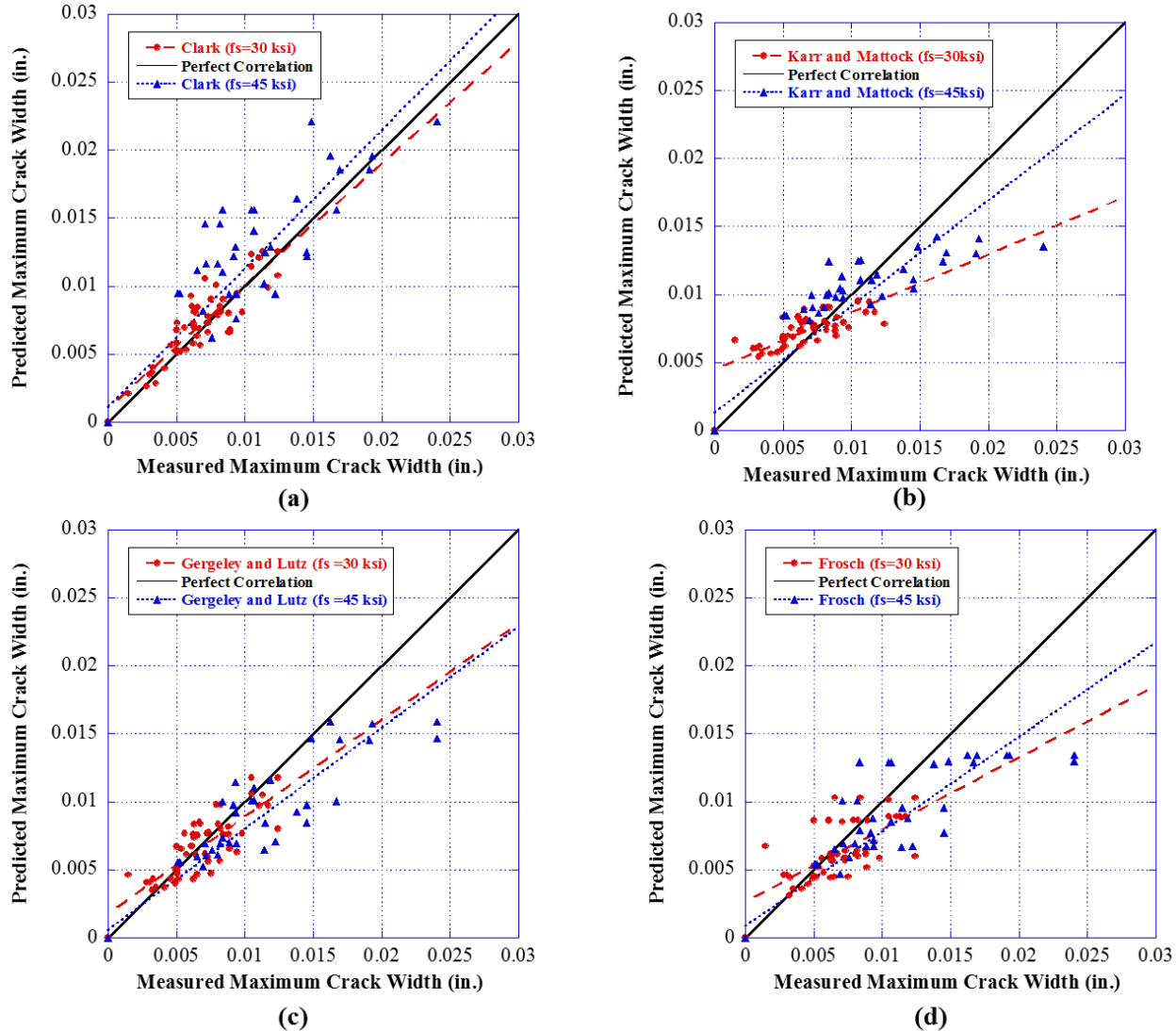


Figure 1. Comparison between the measured and predicted maximum crack width using equation developed by (a) Clark (1956), (b) Karr and Mattock (1963), (c) Gergeley and Lutz (1968), and (d) Frosch (1999)

$$w_c = \frac{2f_s}{E_s} \beta \sqrt{d_c^2 + \left(\frac{s}{2}\right)^2} \quad \text{Eq. 3}$$

where:

- s = maximum permissible bar spacing, in
 w_c = limiting crack width, in. (0.016 in, based on ACI 318-95)
 E_s = Modulus of Elasticity of Steel, ksi
 f_s = Stress in Steel Reinforcement
 β = $1.0 + 0.08 d_c$
 d_c = bottom cover measured from center of lowest bar, in

Rearranging terms in Eq. 3 to solve for f_s as the resistance yields the following:

$$f_s = \frac{w_c E_s}{2\beta \sqrt{d_c^2 + \left(\frac{s}{2}\right)^2}} \quad \text{Eq. 4}$$

The variables E_s , β , and d_c^2 are random variables with statistical parameters. The values w_c and s are deterministic parameters based on ACI318-08 regulations of 0.016in for w_c and $s_{\max} = 12(40000 / f_s)$.

On the other hand, the load Q in the limit state function is the stress in the reinforcement due to the applied loading. Based on sectional analysis and force and moment equilibrium, f_s can be expressed as shown in Eq. 5 below. It is noted that for reinforced concrete beam, the stress in reinforcement can be expressed as a function of the externally applied moment, material and sectional properties as shown in Eq. 5 below:

$$f_s = \frac{3 \left(A_s E_s + 3 E_c b d + \sqrt{A_s^2 E_s^2 + 2 A_s E_s E_c b d} \right) M_{\max}}{d (4 A_s E_s + 9 E_c b d) A_s} \quad \text{Eq.5}$$

where:

- A_s = area of steel reinforcement, in²
 E_s = elastic modulus of steel, psi
 E_c = elastic modulus of concrete, psi
 b = width of concrete section, in
 d = effective depth of concrete section, in
 P = applied load, lbs
 L = span length, in
 $M_{\max}$ = The maximum live load moment.

Thus, the limit state function, $G = R - Q$, can be written as follows:

$$G = \frac{w_c E_s}{2\beta \sqrt{\left(d_c^2 + \left(\frac{s}{2}\right)^2\right)}} - \frac{3\left(A_s E_s + 3E_c b d + \sqrt{A_s^2 E_s^2 + 2A_s E_s E_c b d}\right) M_{\max}}{d(4A_s E_s + 9E_c b d)A_s} \quad \text{Eq. 6}$$

However, Eq. 6 is the limit state function without the consideration of the effects of arching action. Additionally, the empirical method is mainly based on extensive laboratory tests showing that the arching (or compressive membrane) action of the concrete deck could enhance its strength significantly (Figure 2). This method has been validated by extensive nonlinear FE analysis (Fang et al. 1986). The advantage of this design method is that it does not require structural analysis during the design process. The designer could select the area of rebar directly as long as the design conditions specified in Article 9.7.2.4 are satisfied.

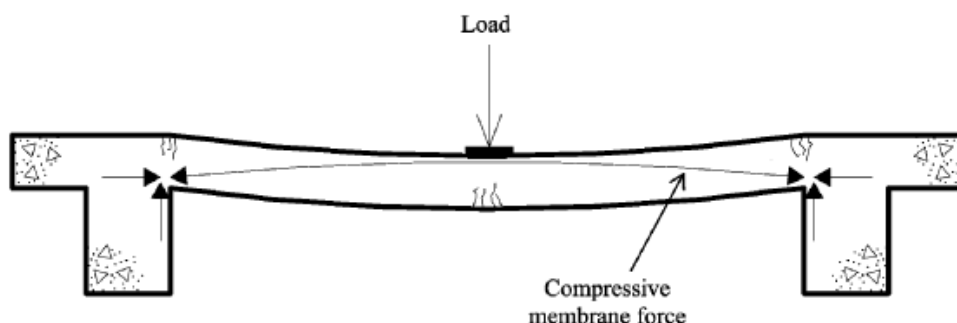


Figure 2. Compressive Membrane Action in Cross Section of Reinforced Concrete Beam-and-Slab Bridge Deck (Hon et al., 2005)

Unlike the reinforced concrete deck designed using traditional method, the moment resistance of the reinforced concrete deck designed using empirical method contains two components: (1) resistance due to tension of the concrete and reinforcement at the bottom fiber of the concrete deck at positive moment region, and (2) resistance due to arching action at the positive moment region. It is noteworthy that the arching action only contributes to the resistance at positive moment areas.

An early study regarding the ultimate strength and arching action effect of concrete was conducted by Park (1964) who proposed a method to analyze the ultimate strength of rectangular concrete slabs based on plastic theory. Many researchers have investigated the effects of arching action on the load carrying capacity and serviceability through experimental study (Tsui et al. (1986), Guice and Rhomberg (1988), Fang et al. (1990^a, 1990^b, 1990^c, 1994), Bailey (2000), Peel-Cross et al. (2000), Taylor et al. (2001), Graddy et al. (2002), Salim and Sebastian (2003), Hon et al. (2005), Taylor et al. (2007), Bailey et al. (2008), and Meadway (2008)). Rankin and Long (1997) performed a series of studies regarding the compressive membrane action and strength enhancement of concrete slabs. They summarized the mechanism of arching action and proposed a method to calculate the strength enhancement.

Thus, for a reinforced concrete deck designed with the AASHTO Empirical Method, the load carrying capacity due to the arching action needs to be considered in the derivation of the resistance model.

Based on study performed by Rankin and Long (1997), the positive moment carried by the arching action with rigid constraint can be expressed as follows:

$$M_{ar} = \frac{M_r 0.85 f'_c d_1^2}{4} \quad \text{Eq. 7}$$

The depth available for arching, d_1 , can be calculated by:

$$d_1 = \frac{D - (p + p_c) \frac{f_{sy} d}{0.85 f'_c}}{2} \quad \text{Eq. 8}$$

Where,

- D = The overall depth of the deck,
- f_{sy} = The yield strength of reinforcement,
- p = The positive reinforcement ratio,
- p_c = The negative reinforcement ratio,
- d = The effective depth of the deck,
- f'_c = Compressive strength of the concrete,

The arching moment ratio, M_r can be obtained by:

$$\text{If } R > 0.26, M_r = \frac{0.3615}{R}; \quad \text{Eq. 9}$$

$$\text{If } 0 < R < 0.26, M_r = 4.3 - 16.1 \sqrt{3.3 \times 10^{-4} + 0.1243R}. \quad \text{Eq. 10}$$

The Geometric and material property parameter for arching, R :

$$R = \frac{\varepsilon_c L_r^2}{4d_1^2} \quad \text{Eq. 11}$$

The plastic concrete strain, ε_c :

$$\varepsilon_c = (-400 + 60 f'_c - 0.33 f_c'^2) \times 10^{-6} \quad \text{Eq. 12}$$

Therefore, after incorporating the effects of arching action, the limit state function for positive moment region can be expressed as follows:

$$G = \frac{w_c E_s}{2\beta \sqrt{\left(d_c^2 + \left(\frac{s}{2}\right)^2\right)}} + \frac{3 \left(A_s E_s + 3 E_c b d + \sqrt{A_s^2 E_s^2 + 2 A_s E_s E_c b d} \right)}{d (4 A_s E_s + 9 E_c b d) A_s} \cdot \left(\frac{M_r 0.85 f'_c d_1^2}{4} - M_{\max} \right) \quad \text{Eq. 13}$$

Since the arching action only affects the positive moment carrying capacity, the limit state function for negative moment region is expressed as shown in Eq. 6.

In order to obtain the maximum moment due to live load applied to reinforced concrete deck and predict the live load moment at various time periods, the Weigh-in-Motion(WIM) data need to be obtained and processed. In this study, since the reinforced concrete deck was designed for axle load, the axle load data including steering axles collected from 32 different WIM sites were processed using a quality control filtering routine established in NCHRP 12-83 (Wassef et al., 2014). The cumulative

distribution functions (CDF) of the processed axle load data was plotted on a normal probability paper. The vertical ordinate of the maximum axle weight, $z_{\max}$, is calculated using Eq. 14 below:

$$z_{\max} = \Phi^{-1}(1/N) \quad \text{Eq. 14}$$

where Φ^{-1} is the inverse standard normal distribution function and N is the total number of trucks.

Therefore, the total number of trucks for the same duration is different for different ADTT, which resulted in different probability as well as different inverse normal distribution value. For illustration purposes, the number of trucks N, probabilities, corresponding inverse normal distribution values z for the corresponding return time period ranging from 1 day to 75 years (for an ADTT of 5000) are shown in Table 2. Table 3 shows the live load models for each ADTT at various return periods. However, to be consistent with previous WIM studies, the live load models were converted to the format of the ratio between the live load effect from WIM data and that from HL-93 load. Table 4 and Table 5 show the bias factors and coefficient of variation (CoV) for various ADTT and return periods.

Table 2. Time Period and Corresponding Inverse Normal Coordinate (ADTT= 5,000)

Time Period	Number of Trucks (N)	Probability (1/N)	Inverse Normal (z)
75 years	136,875,000	7.E-09	5.67
50 years	91,250,000	1.E-08	5.60
5 years	9,125,000	1.E-07	5.18
1 year	1,825,000	5.E-07	4.87
6 months	900,000	1.E-06	4.73
2 months	300,000	3.E-06	4.50
1 month	150,000	7.E-06	4.35
2 weeks	70,000	1.E-05	4.18
1 day	5,000	2.E-04	3.54

Table 3. Statistics for Various ADTTs at Various Return Periods