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Dennis Mertz Symposium on Design and Evaluation of Concrete Bridges

Editors:
Andrzej S. Nowak and Hani Nassif

SP-340

American Concrete Institute
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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

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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-need 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.

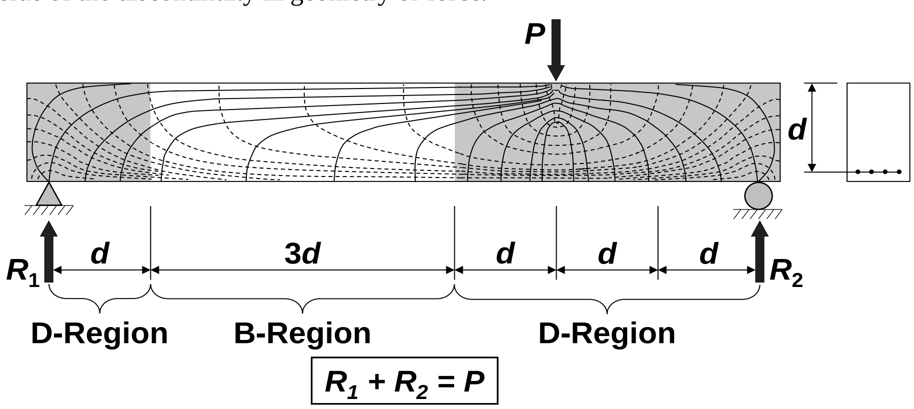


Figure 1—Stress Trajectories within B- and D-Regions of a Flexural Member (adapted from Birrcher [2])

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 Birrcher, 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.

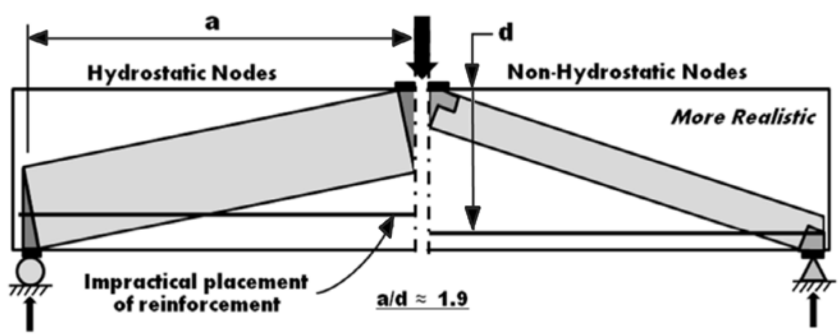
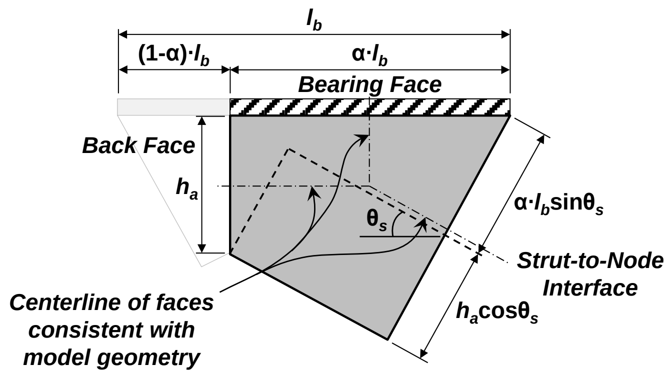


Figure 2—Hydrostatic and Nonhydrostatic Nodes

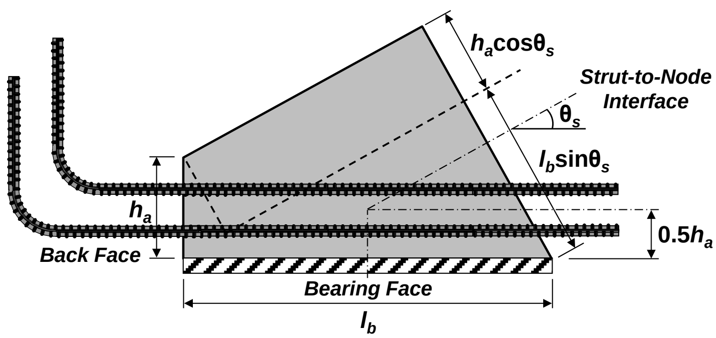
The previous provisions limited the out-of-plane with of concrete that could be included in strut to six diameters of the enclosed bar on each side a stirrup leg, or the cover if less. If there were only two stirrup legs in a given cross section this limited the effective width of a strut to twelve bar 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:

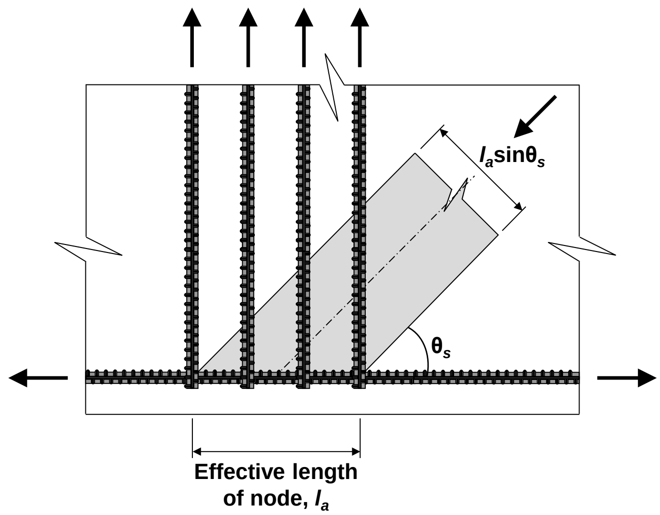
h_a	=	length of the back face of a node
l_a	=	effective length of a CTT node
l_b	=	length of the bearing face
α	=	fraction defining the bearing face length of a portion of a nodal region
θ_s	=	angle between strut and longitudinal axis of the member (degrees)



(a) C*CC* 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.

$f_{cu} = mvf'_c$ (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)

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

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 wthin 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 dhown in Figure 4 (Birrcher [2])

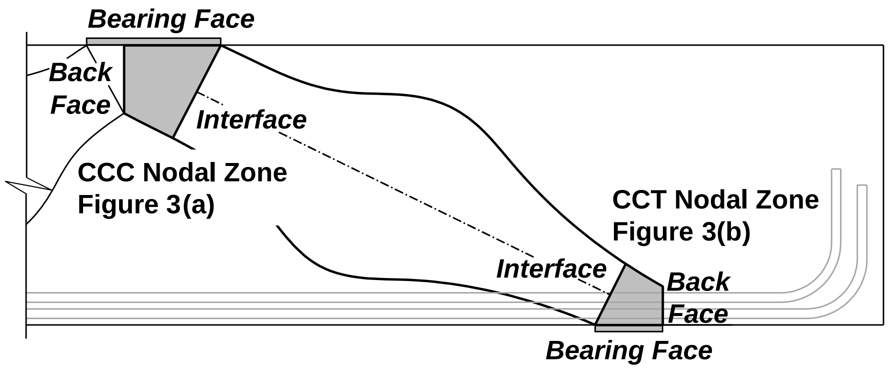


Figure 4— Schematic of bottle-shaped strut

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 pervious 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

Th 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 1.4*Np*, where *Np* 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].

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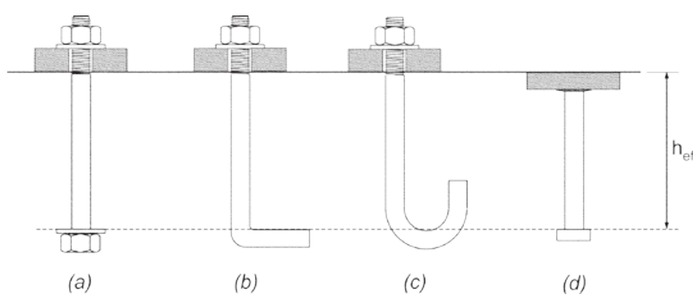
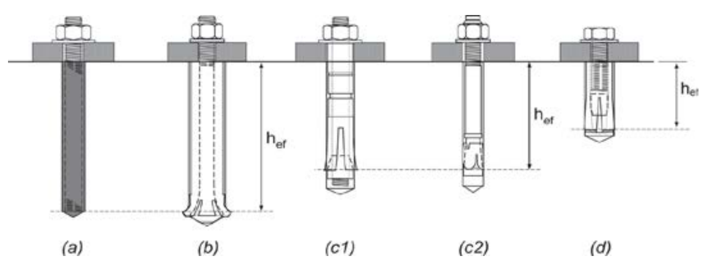


(a) Cast-in-place anchors: (a) hex head bolt with washer; (b) L-bolt; (c) J-bolt; and (d) welded headed stud



(b) Post-installed anchors: (a) adhesive anchor; (b) undercut anchor; (c) Torque-controlled expansion anchors (c-1) sleeve-type and (c2) stud type; and (d) drop-in type displacement controlled expansion anchor

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

<u>Failure Modes and Design Concepts</u>

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

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

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

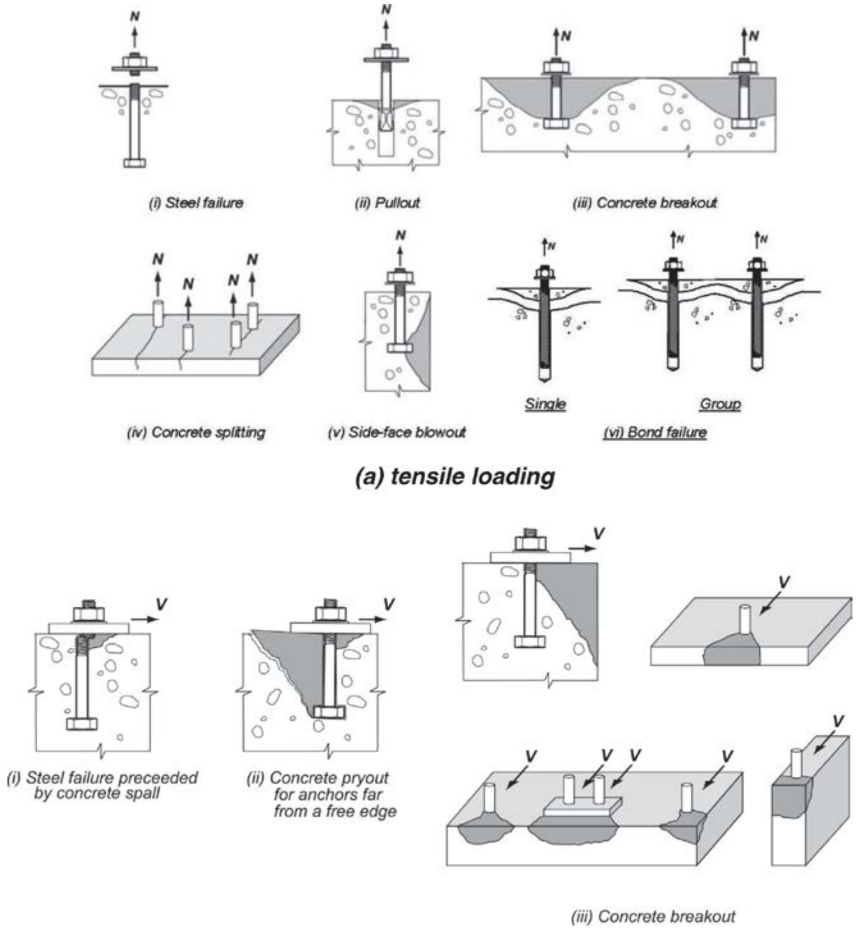


Figure 6—Failure modes considered in ACI 318-14 Chapter 17 (Courtesy of ACI)

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 70ºF or 20 years at 110ºF 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 AAHTO 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, TL, but differ beyond that point. Going from a generalized spectrum to a site-specific elastic seismic response coefficient, Csm, 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 differenced, a comparison shows that AASHTO LRFD Seismic Zone 1 reasonable 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.